GEOLOGIC AND GEOTECHNICAL INVESTIGATION

1925 EL CAMINO DE LA LUZ
SANTA BARBARA, CALIFORNIA

Prepared for:
1729 Calle Boca Del Canon
Santa Barbara, California 93110-2320

OCTOBER 2012

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GEOLOGIC AND GEOTECHNICAL INVESTIGATION
1925 El Camino De La Luz
Santa Barbara, California

For:

Emprise Trust
1925 El Camino De La Luz
Santa Barbara, California 93109

by

COTTON, SHIRES AND ASSOCIATES, INC.
330 Village Lane
Los Gatos, California 95030

October 2012

October 8, 2012
G0058

Emprise Trust
1925 El Camino De La Luz
Santa Barbara, California 93109

SUBJECT: **Geologic and Geotechnical Investigation**
RE: 1925 El Camino De La Luz
(Proposed Single-Family Residence - APN 045-100-024)
Santa Barbara, California

Dear Representatives of the Emprise Trust:

Cotton, Shires and Associates, Inc. (CSA) is pleased to provide Emprise Trust with the following report wherein we describe the findings, conclusions and recommendations of our geologic and geotechnical investigation of the parcel at 1925 El Camino De La Luz (APN 045-100-024, 'the parcel'), Santa Barbara ('the City'), California. It is our understanding that the trust is proposing to develop the parcel with a single-family residence and appurtenant structures. The parcel was previously developed (between 1956 and 1978) with a split-level single-family residence, and is currently (2012) partially developed in conjunction with El Camino De La Luz by a private driveway that jointly serves the parcels at 1925 and 1927 El Camino De La Luz, the City's Mesa Sewer Trunk Line and areas that were graded by the City in 1978, and graded by the neighboring property owner at 1933-1937 El Camino De La Luz.

In the following report, we provide the results of our geotechnical surface and subsurface exploration, monitoring, and slope stability analyses relating to the currently proposed development of the parcel. Included in this report are an engineering geologic characterization and assessment of the site conditions, drainage, and slope stability, and geotechnical design recommendations for residential construction. For clarity, we have provided an Executive Summary at the front of the report to summarize our pertinent conclusions and recommendations.

We appreciate the opportunity to be of service to you on this project. If you have any questions regarding this report, please contact our office.

Very truly yours,

COTTON, SHIRES AND ASSOCIATES, INC.

John M. Wallace
Principal Engineering Geologist
CEG 1923

Patrick O. Shires
Senior Principal Geotechnical Engineer
GE 770

JMW:POS:st

GEOLOGIC AND GEOTECHNICAL INVESTIGATION
1925 El Camino De La Luz
Santa Barbara, California

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EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

This Executive Summary provides a summary of the pertinent conclusions and recommendations resulting from Cotton, Shires and Associates, Inc. (CSA)'s geologic and geotechnical investigation of the parcel at 1925 El Camino De La Luz in Santa Barbara, California. The parcel extends between the Mean High Tide Line on the beach (at elevation +4.63 feet MLLW)[1] and the centerline of El Camino De La Luz (at elevation +140 feet). It is our opinion that development of a single-family residence and appurtenant structures is feasible within the proposed development envelope on the parcel provided that the recommendations of this report are incorporated into the design, construction, and operation of the project during its 75-year economic life.

The potential hazards present at the parcel include static and seismic slope instability, long-term toe and top of bluff retreat, wave run-up, saturation of the weathered Monterey Formation strata, surficial soil creep, and expansive surficial earth materials. CSA analyzed the geotechnical parameters of the site and the static and seismic stability of a representative cross section through the central portion of the parcel. The long-term (1950 through 2010) position of the distinct coastal bluff at the parcel was evaluated by Joseph Scepan Geoscience Consultant (September, 2012), and wave run-up and coastal hazards were evaluated by David Skelly of GeoSoils, Inc. (July, 2012). A detailed discussion of our findings, conclusions and recommendations is presented in the technical report that follows this Executive Summary.

Conclusions

1) The proposed building envelope for the single-family residence and appurtenant structures is located on the south-facing, moderately steep, previously graded and landslide-impacted West Mesa hillside, between elevation 86 feet and elevation 140 feet[1]. Subgrade landform stabilization and residential foundation elements are proposed to extend to approximately 40 to 50 feet below existing grade. No shoreline protective structure is proposed.

2) The pronounced top of coastal bluff is located near elevation 48 to elevation 50 feet, approximately 150 to 160 feet downslope of the southern limit of the proposed building envelope. The coastal bluff face is characterized by a

[1] All elevations in this report are in feet above Mean Lower Low Water (MLLW), unless otherwise indicated.

precipitously (50- to 75-degree inclinations) steep slope that extends down to the toe of the bluff at the rear of the persistent cobble-sand beach near elevation 10 feet. The parcel contains no elevated marine terrace or "upper riser" coastal bluff.

3) The proposed development envelope consists of previously graded landslide debris (maximum of 12 feet thick, at the downslope side of the building envelope at elevation 90 ±5 feet), in-place Monterey Formation bedrock for the majority of the building envelope, and shallow terrace deposits/fill (up to 8 feet thick, at elevation 126 feet to elevation 140 feet). The Tertiary marine sedimentary bedrock materials of the Monterey Formation are characterized by thin-bedded siliceous and calcareous shale with interbedded mudstone. This rock formation is striking (oriented) nearly east-west, and is dipping moderately (generally 20- to 35-degree inclinations) to the south/southwest. The orientation of the Monterey Formation bedrock results in a dip-slope condition with stratigraphic units daylighting in the coastal bluff. The parcel (and eight adjacent parcels to the west and east) experienced reactivated landsliding in 1978, characterized by a deep-seated translational landslide that affected the parcel below elevation 130 feet. The recommended pier and grade beam residential foundation within the proposed development envelope is to be located in intact Monterey Formation bedrock. Shear pins with tiebacks are proposed to increase the slope stability of the building envelope to industry acceptable standards (i.e., factors of safety of 1.5 for static and 1.1 for seismic conditions).

4) CSA's field investigation (2008-2011) included detailed topographic surveying, engineering geologic mapping, small- and large-diameter borehole exploration, piezometer and inclinometer installation and monitoring to facilitate construction of a representative engineering geologic cross section through the parcel.

5) Groundwater levels at the parcel are being monitored using multi-staged vibrating wire piezometers. Using these instruments, we have documented groundwater levels that have fluctuated little over the monitoring period (May 2011 to January 2012), with groundwater depths ranging from 22 to 35 feet below the ground surface.

6) The parcel will likely be subjected to strong, and perhaps violent, seismic ground shaking within the 75-year economic life of the development. The maximum

earthquake events on nearby active faults, such as the Mission Ridge-Arroyo Parida, Santa Ynez, offshore North Channel Slope, Red Mountain, and Oakridge faults, could result in peak ground accelerations at the subject property of up to 0.51g. Based on a probabilistic analysis with an exceedance probability of 10% in 50 years, a peak ground acceleration of 0.44g was determined.

7) No known active or potentially active faults have been mapped across the parcel, and no evidence of faulting was identified at it.

8) The persistent (1850's to 2011) orientation and position of the coastal bluff along the south portion of the parcel is primarily controlled by coastal processes within this subarea of the semi-sheltered Santa Barbara Channel, but has been intermittently impacted by landsliding (in 1978) as described herein. Annualized long-term coastal bluff retreat rates for the Mesa sub-region of the Santa Barbara coastline vary, but have previously been identified to be approximately 4 inches per year (Fisher, 2001), 7.7 inches per year (Norris, 1968), up to 10 inches per year (City of Santa Barbara General Plan, Seismic Safety Element, 1979), and, on a generalized City-wide basis, 12 inches per year during the planning horizon of the recently updated General Plan (City of Santa Barbara, 2011). Historic, long-term (60 year) coastal bluff retreat along this parcel has been analyzed on the basis of aerial photographic imagery between 1950 and 2010 to be at an average annualized rate of 0.8-1.4 inches/year along the toe of the coastal bluff, while as a result of the 1978 landslide and subsequent marine, atmospheric, and anthropogenic processes, the top of the coastal bluff in 2010 is 10.5-33.0 feet further to seaward (south) than it was in 1950 (Scepan, September, 2012).

9) The southerly edge of the proposed residence is located at elevation 92 feet, 150-160 feet upslope of the prominent top of coastal bluff, and thus, 90 to 100 feet landward of the projected coastal bluff retreat range during the 75-year economic life of the residence.[2]

10) Based on our slope stability analysis, we conclude that the proposed residential building envelope, while located upslope of the majority of the 1978 landslide

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[2] Site-specific historical retreat rate (toe of coastal bluff, 1950-2010): (A) 0.8 inches/year x 75 years = 60 inches (5 feet), and (B) 1.4 inches/year x 75 years = 105 inches (8.75 feet). With Coastal Commission's additional 15 feet setback = 20-23.5 feet. The City's adopted generalized coastal bluff retreat rate (without distinction between the toe and top of coastal bluff) is 12 inches/year x 75 years = 75 feet, plus Coastal Commission's 15 feet, = 90 feet from the top of the coastal bluff.

and atop intact Monterey Formation bedrock, currently exhibits a static factor of safety (FS) of 1.39. Residential foundation recommendations are provided herein to improve the building envelope and upslope hillside stability to the industry standards of FS ≥ 1.5 static, and FS ≥ 1.1 to 1.2 seismic (pseudo-static with a seismic coefficient k = 0.15).

11) The State of California has summarized the current standard-of-care for slope stability analysis in Special Publication 117 ("Guidelines for Evaluating and Mitigating Seismic Hazards in California", Department of Conservation Division of Mines and Geology, 1997) and "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California" (June 2002). These guidelines indicate that the acceptable factor of safety (FS) for static conditions is 1.5, and for seismic conditions (pseudo-static analysis using a seismic coefficient of 0.15) varies from 1.0 to 1.2. It is our understanding that the certified City of Santa Barbara Local Coastal Program (LCP) Zoning Ordinance incorporates these State guidelines by reference.

<u>Recommendations</u>

A) The residential building envelope should be stabilized using deep, reinforced concrete piers (shear pins) with tiebacks. We recommend that a shear pin wall with tiebacks be located along the downslope side of the construction envelope, near elevation 90 feet. In addition, an upper shear pin wall, without tiebacks, should be located near elevation 115 feet, but could be incorporated as part of the residential foundation structure. These foundation elements should be designed to resist the anticipated lateral loads imposed by the upslope earth materials, as discussed in Sections 4 and 6, below, in order that the building envelope and area upslope of it achieve static and seismic Factors of Safety of ≥ 1.5 and ≥ 1.1 to 1.2, respectively.

B) The residential structure should be supported on a pier-and-grade beam foundation system with piers embedded a minimum of 20 feet into competent bedrock materials.

C) Because of the detrimental influence of water on stability, erosion, and expansion of earthen material, it is important that surface water be strictly controlled in the project area. We recommend that all surface water within and upslope of the development on the parcel be captured and directed in closed pipe(s) to one or more tanks for beneficial reuse, or in the alternative, be discharged into the City's municipal drainage system. In no event should any water from the proposed development be discharged to, or over (down), the coastal bluff. It will be important that surface and subsurface drainage are strictly controlled to avoid infiltration of rainfall or anthropogenic water to ground (e.g., into any remaining artificial fill, terrace deposits, or near the top of coastal bluff) or into and/or over the coastal bluff. CSA encourages utilization of low stature (fire resistant) deep-rooted native vegetation that does not require substantial irrigation in the residential re-use project landscaping.

D) We recommend installing horizontal drains along the downslope shear pin wall to help prevent a rise in the groundwater levels over those observed during our monitoring period.

E) We recommend that all water fixtures on the parcel be installed with automatic shut-off and back-flow valves, as applicable, and that connecting lateral pipes be located in separate conduits.

F) We recommend that all utility lines and trenches on the parcel, including but not limited to the City's Mesa Trunk Line Sewer, be checked, and upgraded as necessary, to prevent exfiltration of water to the parcel.

G) We recommend that grading at the site be kept to a minimum, and that grading or foundation construction be performed only during the dry season. We recommend that prior to the start of construction, the inclinometers and piezometers be monitored and surface conditions upslope and downslope of the building envelope, and at the coastal bluff, on the parcel be documented.

H) The design drawings and specifications for all proposed (regulatory agency-approved) improvements (development) on the parcel should be reviewed by a Certified Engineering Geologist and Registered Geotechnical Engineer to assure that the recommendations of this report and site-specific design criteria are adequately incorporated into project design and construction.

I) CSA recommends a minimum monitoring and reporting program of conditions at the parcel during the 75-year economic life of the proposed project to document its performance and to inform the property owner and public agencies of any substantial changes in them.

TECHNICAL REPORT

GEOLOGIC AND GEOTECHNICAL INVESTIGATION
1925 El Camino De La Luz
Santa Barbara, California

1.0 INTRODUCTION

This report contains the findings, conclusions, and recommendations of Cotton, Shires and Associates, Inc. (i.e., CSA's) geologic and geotechnical investigation of the parcel located at 1925 El Camino De La Luz, in the City of Santa Barbara (the City), Santa Barbara County, California - APN 045-100-024 (see Location Map, Figure 1). This investigation report is the culmination of a three-year study of the parcel and adjacent areas. It is our understanding that the results of our geologic and geotechnical investigation will accompany focused investigations relating to bluff retreat by Joseph Scepan Geoscience Consultant (September, 2012), and a coastal/wave hazards evaluation performed by David Skelly of GeoSoils, Inc. (July, 2012). In our investigation report, we specifically address the geologic conditions, slope stability, and geotechnical design recommendations for residential development on the parcel.

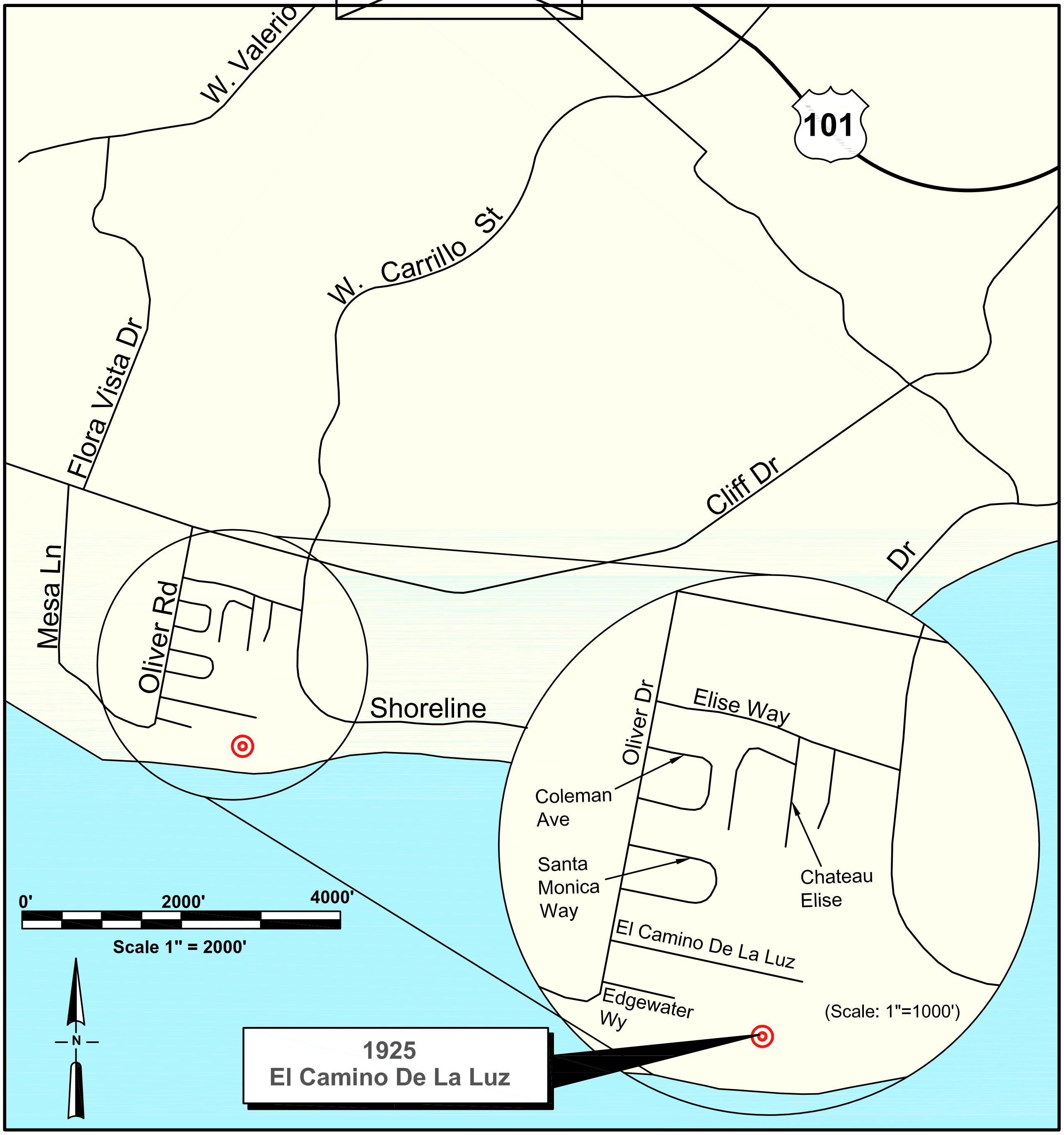
1.1 Proposed Development

It is our understanding that the proposed development is to include a single-family residence and appurtenant structures located in the northerly portion of the 50-foot wide parcel, near the southern terminus of the driveway that is its 'flag' portion. We understand that the base elevation of the building pad is to be at approximately elevation 92 feet (plus or minus approximately 5 feet), and the top elevation of the proposed residence and appurtenant structures will be below elevation 130 feet.

1.2 Background

The ±0.44-acre parcel is presently (2012) developed in various parts with El Camino De La Luz, a connecting driveway that serves both 1925 and 1927 El Camino De La Luz, a segment of the City's combination gravity flow and pumped 1940's vitrified clay pipe Mesa Trunk Line Sewer,[3] other public utility conduits, a minor subareal stormwater v-ditch that discharges to 1921 El Camino De La Luz, older wooden fencing at the

[3] In or about 2006, the City installed an interior lining in the sewer pipe.



Reference: ESRI Base Map, 2011

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

SITE LOCATION MAP

1925 El Camino De La Luz
Santa Barbara, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=2000'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	1

southerly end of the driveway, the City's post-1978 landslide grading envelope, and the Doolittle, et al. 1984 grading envelope.

The parcel was created by City approval of a minor land division (lot split) in 1955 and developed in 1956 pursuant to an inclusive City building permit with a split-level two-story residence and appurtenances.[4] The residence was located on a notched building pad at the top of the headscarp of a pre-existing landslide. The driveway functioned to drain stormwater runoff from the street and other impervious surfaces on adjoining parcels to a drainage ditch along the east side of the house. A large landslide mobilized in the area of 2001 through 1903 El Camino De La Luz between February 11/12 and 14, 1978, during a period of heavy rainfall. Its catastrophic failure on the latter date impacted eight parcels and resulted, among other impacts, in the destruction of the residence and appurtenances south of the driveway at 1925 El Camino De La Luz. The City subsequently removed some, but not all, of the structural debris from the parcel, superficially graded it above elevation 60 feet, and in the process established the terrain that currently exists in that area.

In 1984, grading occurred on 2001-1927 El Camino de la Luz to fill the landslide graben and construct a benched keyway and subdrain-supported buttress on these parcels to help support the residence at 1933 El Camino de la Luz and the Mesa Trunk Line Sewer, and extended through vertical cutting of the coastal bluff top and adjacent hillside onto westerly 1925 El Camino de la Luz. Aerial photography from 1979 to the present indicates that post-1978 landslide coastal processes substantially restored the orientation and position of the coastal bluff, which had been translated seaward, within approximately 20 months. The graded terrain of the parcel has remained essentially unchanged during the subsequent years, including through several substantially above-average precipitation rain years.

A prior geotechnical investigation was performed on the adjacent 1921 El Camino De La Luz parcel in 2006 by Padre Associates, Inc. to analyze the geology and geotechnical conditions of that site in the context of a proposed residence on the upper hillside portion of that parcel. Padres' investigation included the drilling of two 18-inch diameter and three 24-inch diameter boreholes; the three 24-inch diameter boreholes were downhole logged by the project geologist. Laboratory tests were performed, including direct shear testing of downhole samples.

[4] The El Camino De La Luz area of the West Mesa was substantially built out with single-family residences during the 1950's-1970's.

As part of our investigation, we reviewed background documents and maps, including the Padre Associates, Inc. investigation report and comments on it. However, CSA has not relied in our investigation, conclusions, or recommendations on Padres' data, laboratory test results, downhole interpretations, or geologic mapping.

1.3 Purpose and Scope of Work

The objectives of this geologic and geotechnical investigation have been to characterize the site surface and subsurface geologic conditions, determine the stability state of the site, and provide geotechnical design recommendations for construction of the proposed residence and appurtenances. Through exploration of the surface and subsurface geologic conditions and laboratory testing of representative earth materials, the geometry and engineering properties of the surficial earth materials and bedrock materials were characterized to allow our firm to perform detailed slope stability analyses and provide geotechnical design recommendations for construction. To fully address these objectives, we performed the following scope of work:

1. Document Research - Geologic and geotechnical documents, reports, and maps pertinent to the site were obtained, reviewed and analyzed (see Sections 8.1, Documents/Maps and Technical References, for a list of documents, maps and technical publications reviewed).

2. Evaluation of Aerial Photographs - Aerial photographs taken between 1929 and 2005 were analyzed by stereoscopic viewing to identify, within the limitations of the individual photographs, geologic hazards and/or historical changes in topography (see Section 8.2, Aerial Photographs, for a list of aerial photographs evaluated).

3. Topography Surveying – We generated an original, detailed topographic base map of the parcel using our total station theodolite surveying equipment. This topographic survey provides the necessary and accurate base map upon which our engineering geologic mapping was performed, and for generating the topographic profile for use in slope stability analysis, and for evaluating the coastal landforms, including the top and toe of the coastal bluff.

4. Engineering Geologic Mapping – The regional geology was reviewed, and detailed engineering geologic field mapping was performed to determine the site-specific geologic conditions of the subject property. The bluff face and the geologic exposures on the beach at the south end of the parcel were mapped in detail to supplement our surface and subsurface exploration program (see Plate 1, Engineering Geologic Map, in pocket at back of report).

5. Small-Diameter Subsurface Investigation – Three (3), 8-inch diameter borings were excavated, logged, and sampled to depths of 98 to 100 feet on 1925 El Camino De La Luz to determine the subsurface geologic conditions in the subject area. We also excavated and sampled two (2) additional small-diameter boreholes on the adjacent property (1921 El Camino De La Luz), which were used to aid in subsurface characterization of the parcel. These five borings were logged in the field by our staff and principal engineering geologists (see Appendix A, Field Investigation for logs of these borings).

6. Large-Diameter Subsurface Investigation – Two (2), 24-inch diameter borings were excavated, logged, and sampled to depths of 26 to 30 feet on 1925 El Camino De La Luz to determine the subsurface geologic conditions at the parcel. We also excavated and sampled one (1) additional large-diameter borehole on the adjacent property (1921 El Camino De La Luz), which was used to aid in subsurface characterization of the parcel. These three borings were logged in the field by our staff and principal engineering geologists (see Appendix A, Field Investigation for logs of these borings).

7. Instrumentation – We installed three slope inclinometers and three piezometers within the three small-diameter borings on the parcel in May 2011 to help evaluate the groundwater levels at depth, and to assess/characterize any slope movement. We have subsequently regularly monitored these instruments and recorded their data (see Appendix C, Monitoring, for results).

8. Laboratory Testing - Laboratory tests were performed on representative earth materials to obtain critical engineering properties for use in slope

stability analyses and foundation design criteria (see Appendix B, Laboratory Testing, for results).

9. Slope Stability Analyses – Static and seismic slope stability analyses were performed for critical failure surfaces on a representative cross section to evaluate the stability of the site during seismic and static (high groundwater) conditions (see Appendix D, Slope Stability Analysis Results).

10. Geotechnical Analyses and Formulation of Recommendations – Research, field, laboratory, and slope stability data were analyzed to characterize the geologic and geotechnical site conditions. The conclusions and recommendations set forth in this report were developed from these analyses.

11. Presentation of Findings - This report and the accompanying illustrations were prepared to summarize the findings, conclusions and recommendations of our investigation.

2.0 PHYSICAL SETTING

The parcel at 1925 El Camino De La Luz is approximately 344 feet in length, 50 feet in width along its main 'flag' body, and has a 200-foot long by 12.5 feet wide 'flag pole' lot component along the driveway and to the centerline of the street. The southern, downslope property line of the parcel is along the Mean High Tide Line, approximately 32 feet seaward of the toe (lower break in slope) of the coastal bluff. The parcel is bounded to the east and west by single-family residences on variously sized parcels, to the south by the Pacific Ocean (Santa Barbara Channel), and to the north by El Camino De La Luz. The physical parameters that influence the parcel include topography (terrain), the geologic setting, seismicity, coastal processes, and surface and subsurface hydrology. In the following report sections, we present descriptions of each of these parameters, including discussions of the influence that each parameter has on the subject area. Skelly (2012) describes the coastal processes that impact the parcel.

2.1 Topography

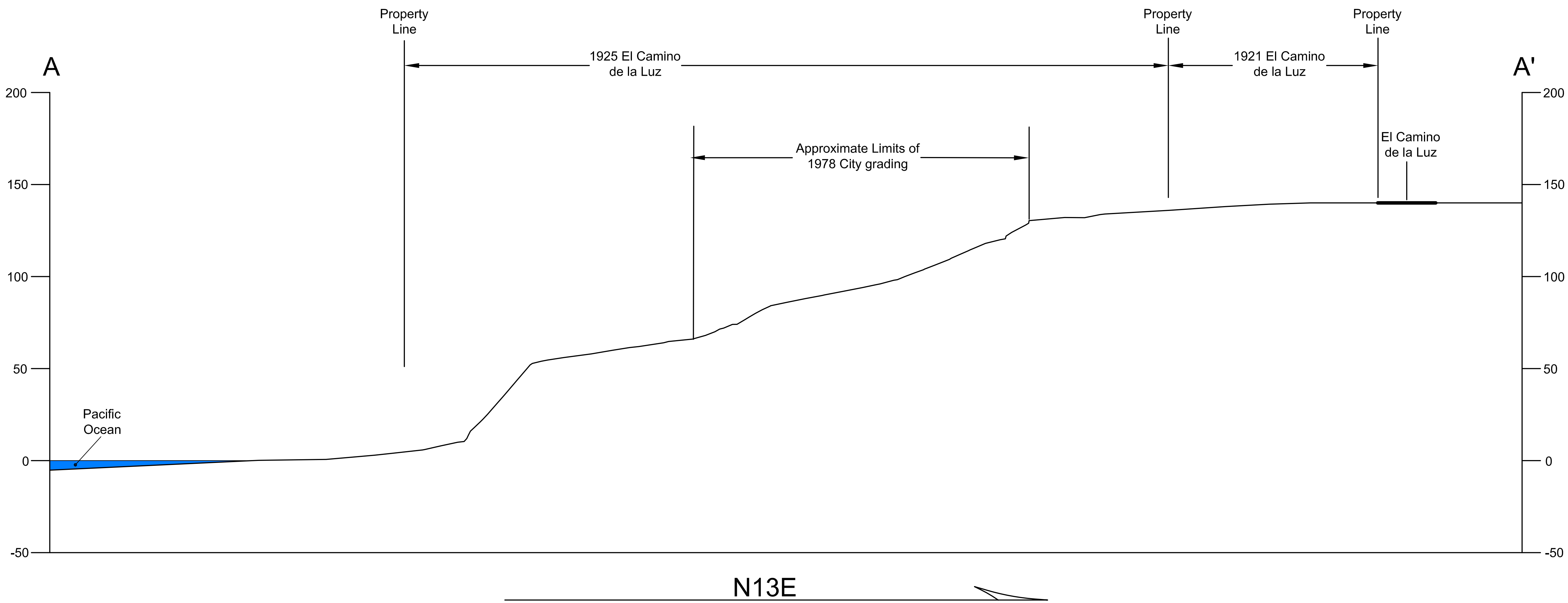
2.1.1 Terrain - The terrain at the parcel, at present (2012), consists of a gently inclined to moderately steep (5- to 25-degree inclination), south-facing, hillside, distinct coastal bluff, and rear cobble-sand beach on a gently inclined wave-cut terrace that extends seaward from the toe of the coastal bluff. Elevations on the parcel range from el. 4.63 feet along the seaward Mean High Tide Line property line of the parcel, to elevation 140 feet at El Camino De La Luz. The proposed base floor elevation of the residence is to be approximately elevation 90 feet (plus or minus 5 feet). The distinct top of coastal bluff is located near elevation 48 to 50 feet, approximately 150 to 180 feet downslope from the southern limit of the proposed building envelope. The coastal bluff is characterized by a precipitously steep slope (50- to 75-degree inclinations) that extends, with localized variations, down to the beach near elevation 10 feet, where a prominent break in slope is defined by a persistent cobble/boulder layer on the back beach. At the toe of coastal bluff immediately above the cobbles, marine erosion has developed a low height and narrow horizontal feature along the easterly half of the parcel and a small cave has developed along a near-vertical fracture zone near a Monterey shale outcrop on the adjacent 1927 El Camino De La Luz parcel. Approximately 25 feet to the east of the 1925 El Camino De La Luz parcel, a relict drainage swale on 1921 El Camino De La Luz cuts through the top of the coastal bluff and contributes to minor bluff face erosion in the area of a small promontory.

2.1.2 Top of Bluff - The southern end of the hillside on the parcel contains a very steep to precipitous (50- to 75-degree inclinations) coastal bluff. The coastal bluff face, with localized rilling, extends downslope to where the toe of the bluff face intersects the beach at approximately elevation 10 feet. The height of the coastal bluff from the well-defined top to the toe is 38 to 40 feet, and approximates the pre-1978 landslide grade toward the relict drainage swale on adjacent 1921 El Camino De La Luz parcel. The intersection of the top of the precipitous coastal bluff face with the relatively more gently inclined hillside to the north is a highly conspicuous, well-defined "top of bluff" point on the topographic profile (see Figure 2). The parcel contains no elevated marine terrace or "upper riser" coastal bluff feature. A representative topographic profile is illustrated on Figure 2 through the central portion of the parcel. This profile was generated from our detailed topographic surveying in 2010. Recent (2012) site reconnaissance indicates that the site topography (terrain) has not experienced significant changes between the time of our survey and the present.

Our evaluation of the site topography reveals that this prominent coastal bluff is located near the southern edge of the subject property. This coastal bluff is characterized by: 1) a prominent toe of slope at the back beach; 2) a precipitous bluff face, the lower portion of which is subject to episodic (limited period) wave run-up during high surf; and 3) a distinct top of the coastal bluff defined by a prominent inflection in the bluff face where the precipitous bluff face abruptly transitions to a more gently inclined to moderately steep bluff top hillside. This top of coastal bluff location is consistent with the methodology for identifying (determining) the top of coastal bluff as set forth in Coastal Commission's adopted regulation that defines this term of art (14 Cal. Code of Regs., sec. 13577), Coastal Commission's "Geologic Stability of Blufftop Development" Guideline, and Coastal Commission staff geologist Mark Johnsson's paper on "Establishing Development Setbacks from Coastal Bluffs" (2002). Johnsson defines the top of coastal bluff as:

"The bluff edge is simply the line of intersection between the steeply sloping bluff face and the flat or more gently sloping bluff top."

The parcel, as illustrated by the topographic profile of Figure 2, clearly has a well-defined top of coastal bluff approximately 288 feet south of the east-west property line with the adjacent 1927 El Camino De La Luz parcel. This top of coastal bluff (including the fact that it was lowered by grading in 1984) is consistent in its trend across the parcel and adjacent parcels, and has been in this approximate position and in this form dating



Horizontal Scale = Vertical Scale

NOTES:
1) Topographic Profile surveyed by Cotton, Shires and Associates, on April 15 and April 16, 2010.
2) Mean Lower Low Water (MLLW) data obtained from National Oceanic and Atmospheric Administration (NOAA), April 2010 tidal predictions for the Santa Barbara Area.
3) MLLW datum is 2.788' lower than Mean Sea Level (MSL)

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Topographic Profile A-A'
1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
JD	1"= 50'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	2

back to the earliest available aerial photographs taken in 1928. The 1978 landslide pushed this top of coastal bluff seaward approximately 80 feet, but the top of coastal bluff was restored to near its original position within several years of the landslide occurrence. Scepan (September, 2012) indicates that the top of coastal bluff remains approximately 10 to 33 seaward of its 1950 position. The proposed location of the residential structure must demonstrate that it is inland of the 75-year anticipated retreat rate setback line from the top of the coastal bluff, and maintain suitable static and pseudostatic (F.S.=1.5 and 1.1, respectively) factors of safety. It is our opinion that the proposed building envelope is well inland of the 75-year anticipated retreat rate setback line from the well-defined top of bluff, and the proposed building site would be founded in (on) ground that has not experienced prior landslide movements, and should maintain a suitable factor of safety provided that the foundation design recommendations of this report are incorporated into the project.

2.1.3 Bluff Retreat – The orientation and location of the coastal bluff in the Mesa subarea of the Santa Barbara coastline that includes the subject parcel has demonstrated long-term persistence during the past 60 to 80+ years (Scepan, 2012; GSI, 2012). Previous studies and adopted reports have identified a general shoreline retreat rate (annualized average retreat) of 7.7 inches/year (Norris, 1968, reflected as 8 inches/year in the City of Santa Barbara LCP land use plan [1978, 2004]), and up to 10 inches/year (City of Santa Barbara General Plan Seismic Safety Element 1979). Fischer (2001) and Campbell (2007) have identified coastal bluff retreat rates of approximately 4 inches/year at a site some 500 feet to the east of the subject parcel, and on the adjacent parcel to the west. The City's recently adopted General Plan (2011) posits a generalized coastal bluff retreat rate of 12 inches/year. However, the site-specific study of the location of the coastal bluff top (edge) indicates that as a result of the seaward distention of the landform associated with the anthropogenically activated 1978 El Camino de la Luz landslide, the top of coastal bluff at 1925 El Camino de la Luz was located 10 to over 33 feet further south as of 2010 than it was in 1950 (Scepan, 2012).

The location of the top of coastal bluff on the subject parcel has not measurable changed during the past two years. Concurrently, the toe of coastal bluff has retreated at 1925 El Camino de la Luz at a long-term annualized average of 0.8 inches/year to 1.4 inches/year at the respective easterly and westerly parcel boundaries (Scepan, 2012). The proposed residential reuse development envelope on the subject parcel is set back a minimum of 171.9 feet from the top of the coastal bluff (GSI, 2012), landward of both the City's most recently adopted regulatory coastal bluff setback line (including Coastal Commission's additional 10-foot setback) and 68 to 73 feet from the coastal bluff in 2087, under the City's presently pending tiered (accelerating) coastal bluff retreat rate in the Climate Action Plan.[5] The proposed project therefore meets and exceeds all applicable coastal bluff retreat setback standards of the City of Santa Barbara.[6]

2.2 Geologic Setting

The parcel at 1925 El Camino De La Luz is located along the southern flank of the Santa Ynez Mountain Range in the central portion of the western Transverse Ranges of California. The western Transverse Ranges extend for approximately 75 miles from Point Arguello to Ventura County, and are generally characterized by tightly folded, Mesozoic and Cenozoic marine and non-marine sedimentary strata. Regional compressive stress within the late Tertiary in a north-south orientation has resulted in a prominent mountain building event, producing the characteristic east-west structural grain of the Transverse Ranges. The southern flank of the Santa Ynez Mountains contains early to mid-Tertiary sedimentary strata, tightly folded and extensively faulted along a nearly east-west trend (see Figure 3, Regional Geologic Map, by Minor, 2003).

Mapped active and potentially active faults are mostly high-angle reverse faults, and historic seismicity is characterized primarily by reverse fault focal mechanisms. Rapid uplift of the range from the Pliocene to recent time, which ranges from 1mm/yr to 2mm/yr (Keller, 2000), has resulted in rapid downcutting, deeply incised drainages, and abundant mass wasting. Large fanglomerate deposits are located at the base of the range and are locally blanketed by uplifted marine terrace deposits and alluvium. Landslide deposits are common along the southern flank of the Santa Ynez Mountains, with the fine-grained Tertiary deposits particularly susceptible to landsliding.

The subject site is located in the Mesa area of the City of Santa Barbara, which is characterized as a Pleistocene uplifted marine terrace seaward of the southern flank of

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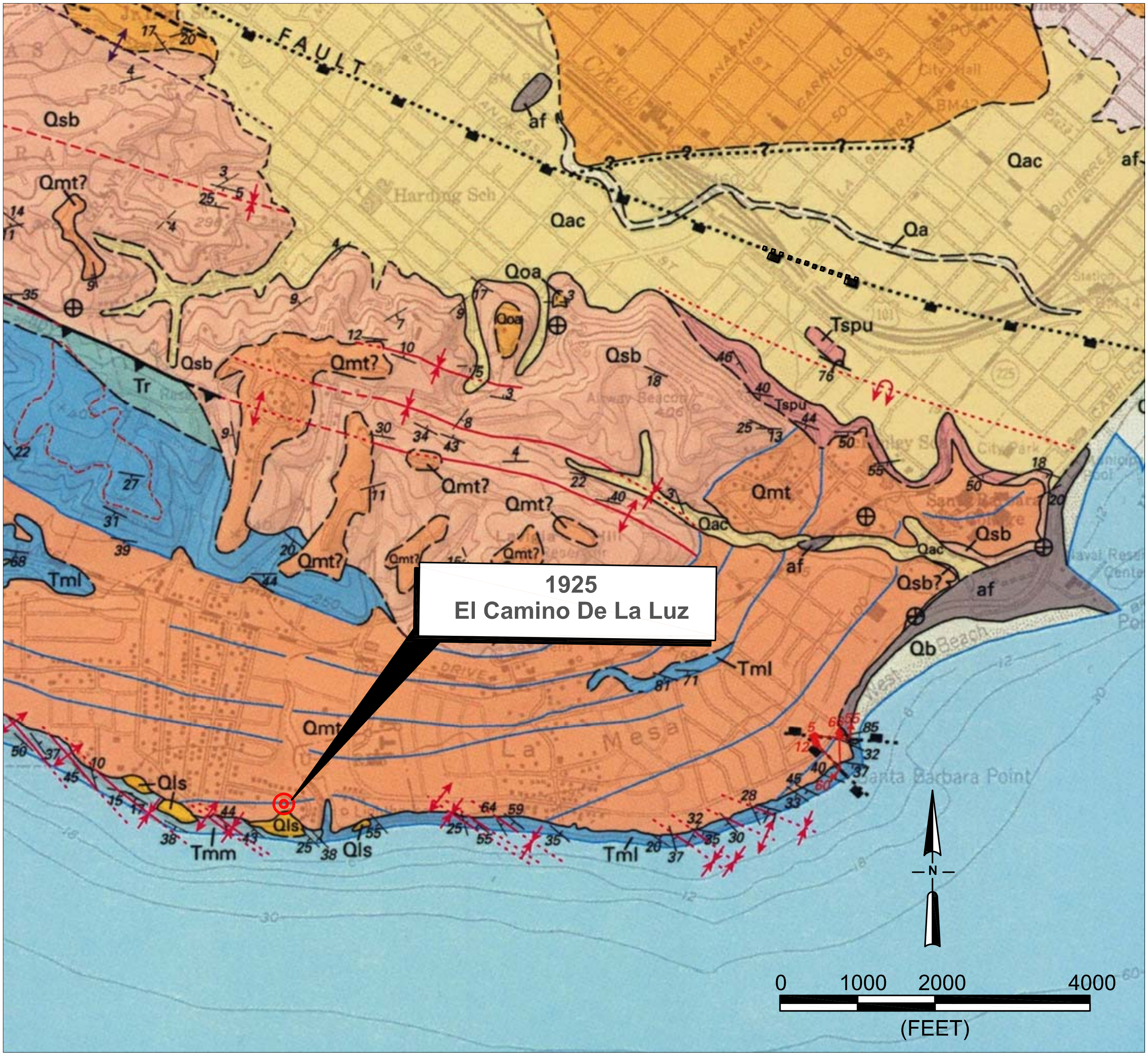
5 The City's pending draft Climate Action Plan (June, 2012) proposes an annualized average coastal bluff retreat rate of 12 inches/year through the year 2050 and a rate of 24 inches/year for 2051-2100. CSA will confirm the location of the coastal bluff by a supplemental site-specific survey prior to submittal of the residential reuse project permit application to the City.

6 At 75-100 years, with an additional 10-ft wide band as applied by the California Coastal Commission within its jurisdiction, the setback dimensions are 85 to 110 feet from the top of the 2010 coastal bluff. Although no measurable top of coastal bluff erosion has been identified on this parcel since CSA's 2010 topographic survey, CSA can confirm the location of the top of coastal bluff by a supplemental area-specific survey prior to finalization of the residential re-use project permit application package to the City.



Reference: Geologic Map of the Santa Barbara Coastal Plain Area, Santa Barbara County, California, Minor et al., 2009

EXPLANATION

GEOLOGIC UNITS

Qls	Landslide deposit
Qmt	Marine Terrace Deposit
Qsb	Santa Barbara Formation (sandstone)
Tmm	Miocene Shale
Tml	Miocene Limestone
Tr	Rincon Shale

SYMBOLS

53 Strike and Dip

Reverse Fault

Thrust Fault

Axis of Syncline or Anticline

Marine-Terrace Shoreline Angle

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

REGIONAL GEOLOGIC MAP

1925 El Camino De La Luz
Santa Barbara, California

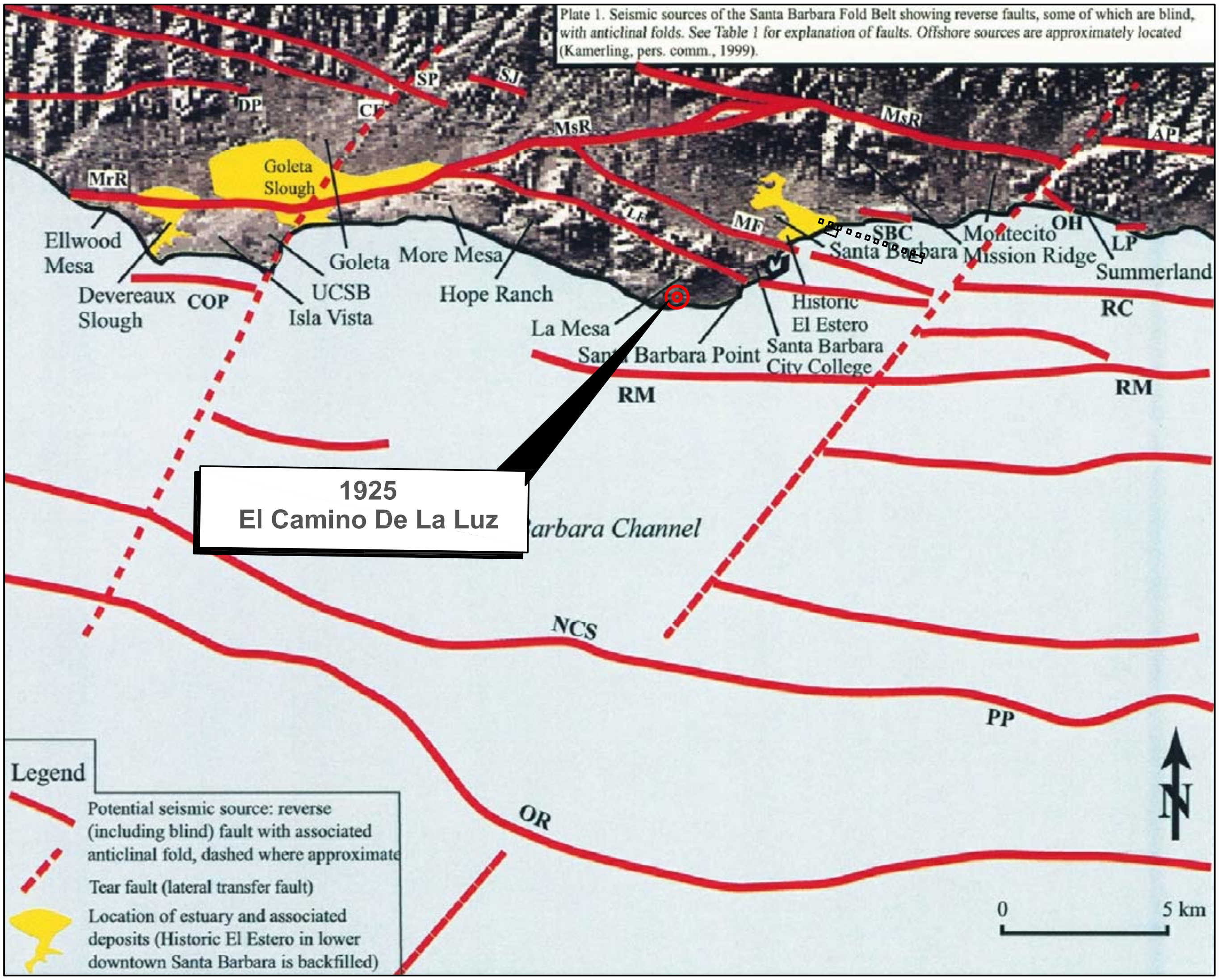
GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=2000'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	3

the Santa Ynez Range. The parcel is located at approximate Latitude 34° 23′ 47′′ and Longitude 119° 43′ 30′′, approximately 780 feet west of the Santa Barbara Lighthouse. The site is underlain by Tertiary marine sedimentary bedrock materials of the Monterey Formation. This rock formation is oriented nearly east-west, and is dipping moderately (20- to 35-degrees) to the south/southwest. The Monterey Formation in the general vicinity is typically characterized by well-bedded, thin-bedded siliceous and calcareous shale with isolated dolomitic concretionary beds, and thin volcanic interbeds that are commonly altered to bentonite. Residual and colluvial soil materials derived from the Monterey Formation can be prone to expansivity and soil creep. No elevated scarp or 'upper riser' of a second marine terrace occurs on the subject parcel.

2.3 Seismic Setting

The Santa Barbara coastal plain and the Santa Ynez Mountains are located in a very seismically active area. Historically, this area has been subjected to strong seismic ground shaking from major earthquakes, and will continue to experience strong ground shaking in the future. The project site is located near several potentially active and active faults, some of which have ruptured in historic times. The faults in the region are mostly reverse slip faults, and are capable of generating strong seismic ground shaking. Some of these faults include the Mission Ridge-Arroyo Parida fault, the Santa Ynez fault, the offshore North Channel Slope fault, and the Red Mountain fault. The Lavigia fault is the closest (1.0 mile) mapped fault to the site (see Figure 4, Regional Fault Map, Keller, 2000); however, due to the lack of Holocene activity and relatively short fault length (15 km), this fault is not the design fault for this site. The active San Andreas fault zone is located approximately 41 miles to the east of the site, and while it represents the fault with the highest activity level and potential for large earthquakes, its distance precludes it from being a significant seismic constraint to the site.

The site has experienced strong ground shaking from several large earthquakes in the historic past. Most notably, the offshore Santa Barbara Channel faults have ruptured several times in the historic past, including a M7.0 in 1812, M6.25 in 1883, M6.3 in 1925, M5.9 in 1941, and M5.1 in 1978. The Santa Barbara earthquake of 1925 damaged or destroyed numerous buildings in the City's commercial district and resulted in 13 deaths. Large earthquakes on the San Andreas fault have also been experienced in the area, including the 1812, 1857, and 1906 multi-segment major earthquakes, and the Parkfield earthquakes of 1934, 1952, and 1966.



Reference: Earthquake Hazards of the Santa Barbara Fold Belt, California, (Report) E.A. Keller, 2000

EXPLANATION

FAULTS

MrR	More Ranch	RM	Red Mountain
MsR	Mission Ridge	OR	Oakridge
AP	Arroyo Parida	NCS	North Channel Slope
DP	Dos Pueblos	RC	Rincon Creek
CF	Carneros	PP	Pitas Point
SJ	San Jose		
SP	San Pedro		
LF	Lavigia		
MF	Mesa		
OH	Ortega Hill		
LP	Loon Point		

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

REGIONAL FAULT MAP

1925 El Camino De La Luz
Santa Barbara, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	As noted	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	4

2.3.1 Deterministic Seismic Analysis – The site could be affected by strong seismic ground shaking due to an earthquake from one of several active faults in the region. The Mission Ridge-Arroyo Parida fault is the active fault that poses the greatest potential threat to the site from a seismic shaking standpoint, and a maximum probable earthquake on this fault should serve as the "design event". The faults tabulated below could produce strong seismic shaking and are all considered to be active or potentially active. The following table provides the results of our deterministic analysis and lists the major earthquake sources, distance from the source to the site, maximum moment magnitudes and peak horizontal ground accelerations anticipated at the site:

TABLE 1 – DETERMINISTIC SEISMIC ANALYSIS

Fault Source	Distance (mi.)	Moment Magnitude[1]	Peak Horizontal Accelerations (g)[2]
Mission Ridge/ Arroyo Parida	4.8	7.2	0.51
North Channel Slope	5.7	7.4	0.51
Red Mountain	4.7	7.0	0.50
Oak Ridge-Mid Channel	8.1	6.6	0.34
Santa Ynez (E)	8.6	7.1	0.30
Santa Ynez (W)	8.8	7.1	0.30
San Andreas (1857)	41.4	7.8	0.10

[1]Based on “Probabilistic Seismic Hazard Assessment for the State of California” by CDMG, DMG Open File Report 96-08.

[2]Based on attenuation relationships developed by Sadigh, et al., 1997 (Horiz. - Rock) as determined using the computer program EQFAULT by Blake, 1989, updated 1997.

Using the maximum probable earthquake (which is considered to be the maximum earthquake expected to occur in a 100-year period) and a **deterministic** approach based on an attenuation relationship developed by Sadigh, et al. (1997), the subject site is expected to experience a mean peak horizontal ground acceleration of 0.51g. Based on a **probabilistic** analysis with an exceedance probability of 10% in 50 years, a peak ground acceleration of 0.44g was determined (U.S.G.S., 2011).

3.0 SITE CONDITIONS

The site geologic conditions of the subject parcel were investigated by our engineering geologists in order to characterize the surface and subsurface materials so that appropriate geotechnical design recommendations could be developed. The surface conditions were characterized using detailed topographic surveying and profiling followed by detailed engineering geologic mapping, which is portrayed on our Engineering Geologic Map (see Plate 1 and Figure 5, reduced version of Plate 1). The site subsurface conditions were investigated using small- and large-diameter boreholes, and these conditions are portrayed on our Engineering Geologic Cross Section A-A' (Figure 6). The groundwater conditions were characterized by the installation of multi-staged piezometers in each of the three small-diameter boreholes on the parcel. Inclinometers were also installed within each small-diameter borehole on the parcel to verify current slope stability and as a means of verifying future slope stability.

3.1 Surface Conditions

The parcel surface is developed, in parts, with the driveway and adjacent southerly half of El Camino De La Luz (ECDLL), weedy and horticultural landscaping east of the driveway, a minor stormwater drainage v-ditch that discharges to the parcel at 1921 El Camino De La Luz, an older wooden fence (with seasonal sandbags to divert subareal runoff to the v-ditch) at the base of the driveway, a chain link metal fence that extends easterly near the top of the 1978 landslide headscarp, erosion control shrubs, the City's 1978 and the 1984 Doolittle, et al. grading envelopes. A vigorous contiguous stand of lemonade berry has colonized the landslide debris, colluvium, and (1984) excavated slope on the parcel below elevation 70 to 80 feet from a small number of shrubs that were repositioned by the 1978 landslide and subsequent City grading. Southern coastal bluff scrub substantially covers the distinct coastal bluff except at lower elevations that experience episodic wave spray or where the colluvium and landslide debris mantle has been eroded. The back beach area of the parcel consists of a persistent cobble field, with seasonal sand lenses and episodic marine detritus. In the biological reconnaissance report of 1925 El Camino De La Luz by WRA, Inc. (2012), the flora and fauna on the parcel are further described and analyzed.

The northern "flag pole" portion of the parcel is relatively level (less than 5-degree inclinations) and underlain by unconsolidated alluvial terrace deposits and artificial fill. The northern "flag" portion of the 50-foot wide area of the parcel, located just south of

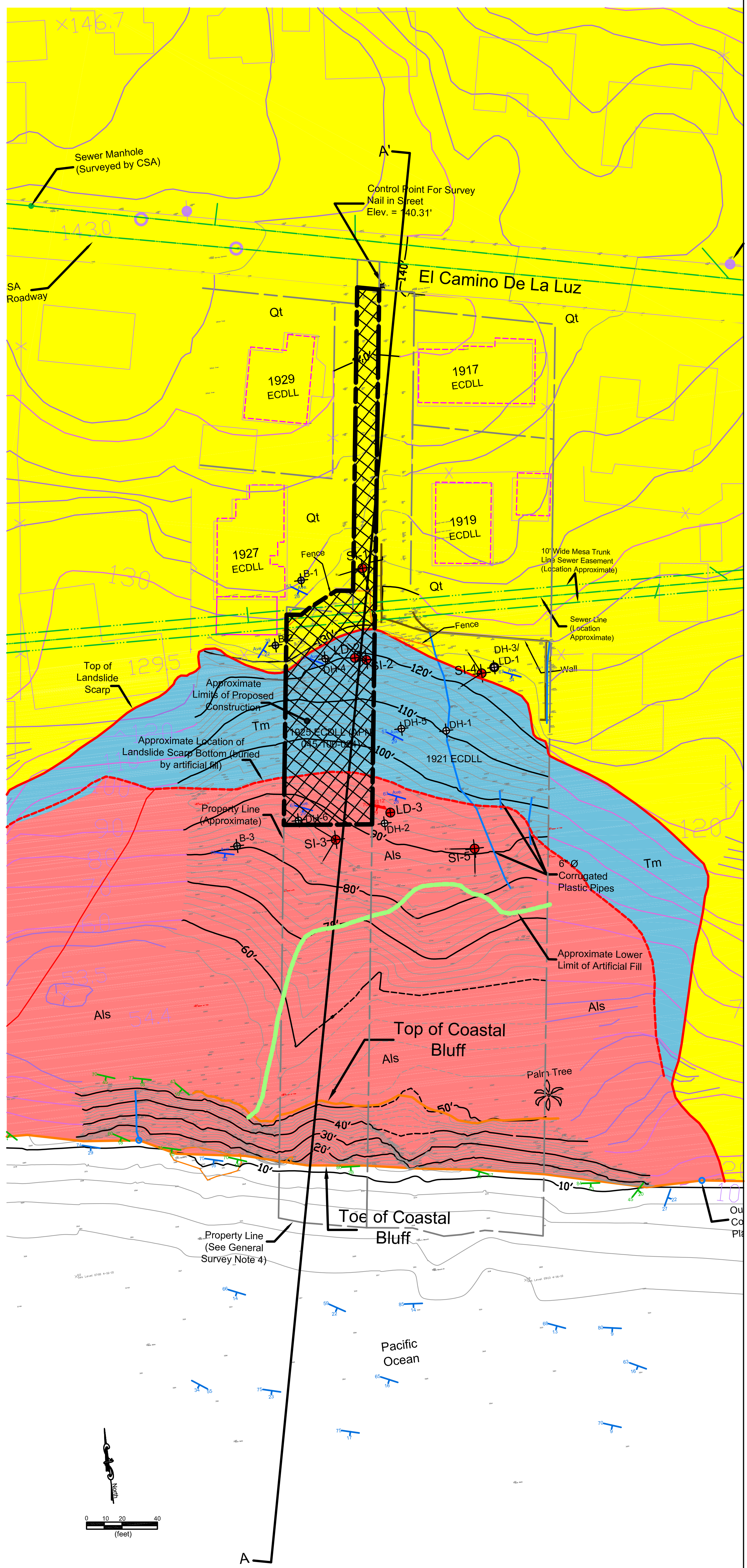
EXPLANATION

Earth Materials

- Qt — Quaternary Terrace Deposits
- Tm — Monterey Formation
- Als — Active Landslide

Map Symbols

- SI-5 — Small-diameter boring with inclinometer casing and piezometers installed by Cotton, Shires and Associates, Inc. in May 2011
- LD-3 — Large-diameter boring drilled and logged by Cotton, Shires and Associates, Inc. in October 2011
- DH-6 — Large-diameter boring drilled by Padre Associates, Inc. in September 2005 (DH-1 and DH-2) and June 2006 (DH-3 through DH-6)
- B-3 — Large-diameter boring drilled by Campbell Geo, Inc. in December 2006
- Stratigraphic bedding orientations collected by Cotton, Shires and Associates, Inc.
- Average stratigraphic bedding orientations collected during the logging of large-diameter borings
- Bedding orientation on landslide basal rupture surface collected in large-diameter boring
- 70′ — CSA 10' Contour
- CSA 2' Contour
- CSA 10' Contour (Approximate)
- CSA 2' Contour (Approximate)
- CSA Survey Point
- 60 — City of Santa Barbara 10' Contour
- City of Santa Barbara 2' Contour
- All Lines of This Color Indicate Features From City of Santa Barbara Map, Including, but not Limited to: Houses, Fences, Roads, Vegetation and Power Poles



SURVEY LIMITATIONS NOTES

1. This is not a map of a boundary survey. No property corners have been set as part of this work.

2. Survey monuments found in the course of this mapping are set by others, and have been used only as a reference for the purpose of topographic mapping, without our verification of their agreement with applicable legal descriptions and seniority of deeds.

3. Relation of topographic features (i.e., fences, walls, trees, power poles, etc.) to property lines as shown on this map is subject to the adjustments that a boundary survey may require.

4. This survey was prepared without the benefit of a Title Report. Easements, if shown, should be considered approximate in location.

5. If this map is provided in an electronic format as a courtesy to client, delivery of the electronic CAD file does not constitute delivery of a professional work product. The signed paper print delivered with this electronic CAD file constitutes our professional work product and, in the event the electronic CAD file is altered, the print must be referred to for the original and correct survey information. We shall not be responsible for any modifications made to the electronic CAD file or for any products derived from the electronic CAD file which are not reviewed, signed and sealed by us.

General Survey Notes

1) All dashed lines on this map represent features (houses, walls, topography, etc.) that have not been surveyed by Cotton, Shires and Associates and are approximate only.

2. Vertical Datum for CSA topography based on NOAA published value for mean lower low water (MLLW) in Santa Barbara.

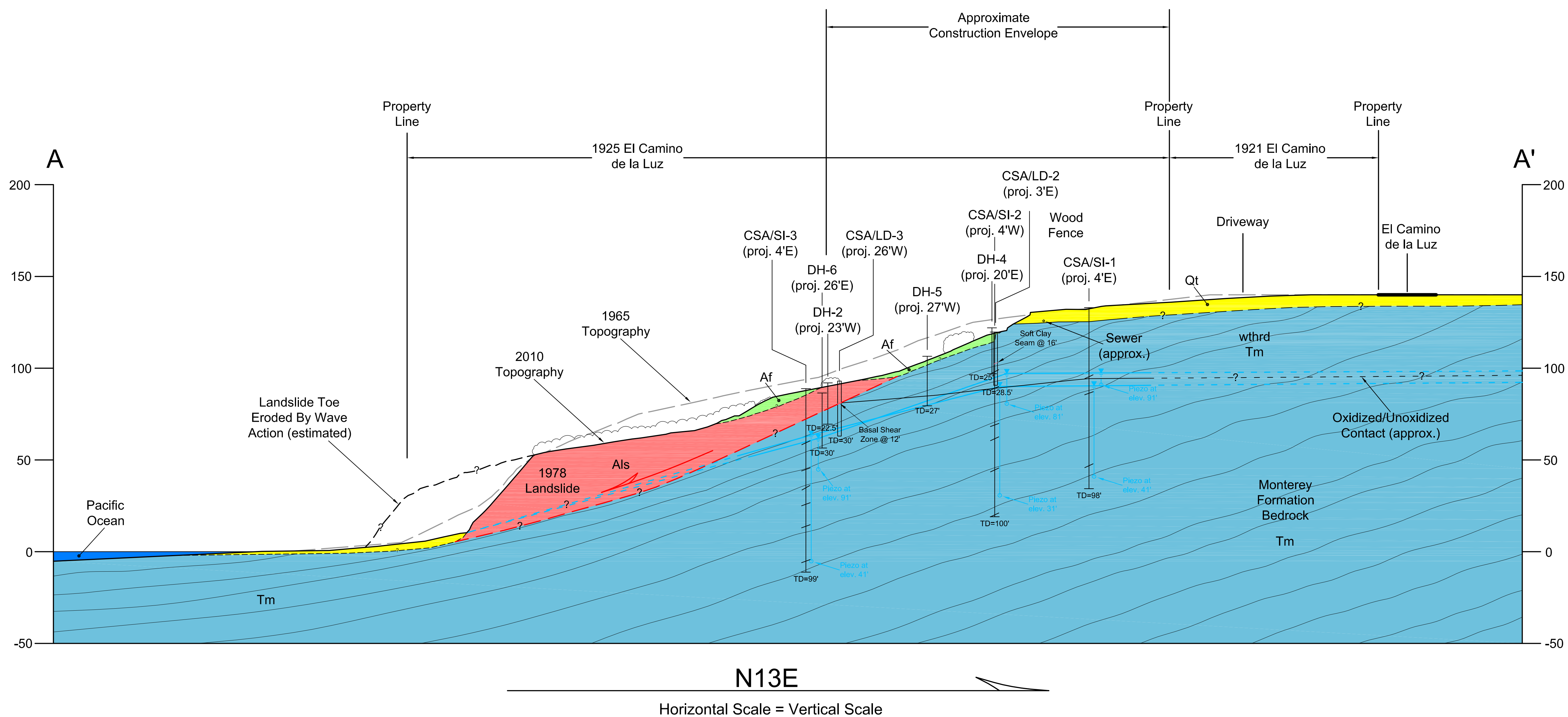
3. City of Santa Barbara topography and features taken from map dated 4/10/95 (Revised April 1997) from County of Santa Barbara website (http://www.countyofsb.org/pwd/water/TopoFloodControl1.htm).

4. Southern property lines are based on the MHTL elevation of 4.63 feet above MLLW (MHTL from David Skelly, GeoSoils, Inc., "Wave Runup & Coastal Hazard Analysis, 1921 El Camino de la Luz & 1925 Camino de la Luz, Santa Barbara, Santa Barbara County, California").

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Site Plan and Geologic Map
1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY AM/TPS	SCALE 1"= 60'	PROJECT NO. G0058
APPROVED BY JW	DATE OCTOBER 2012	FIGURE NO. 5



Cotton, Shires and Associates, Inc.
Consulting Engineers and Geologists

Engineering Geologic Cross Section A-A'
1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
JD	1"= 60'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	6

the driveway and the site of the proposed residence, is characterized by moderately steep (20- to 25-degree inclination) hillside topography. This area was graded for development of the original residence in 1956, again in or about August-September, 1978, following the February 1978 landslide, and along its westerly margin in 1984. The 1978 grading resulted in a relatively uniformly sloping surface topography between elevations 130 feet and approximately 80 feet. This grading resulted in a loosely graded mantle of landslide earthen debris and colluvium, with numerous voids and relict structural debris between elevations ~60 feet and 80 to 90 feet. The 1984 grading by Doolittle, et al. resulted in a cut along the westerly margin of the parcel, between the top of coastal bluff and elevation 90 feet by approximately 6 to 10 feet vertical. Colluvium and landslide debris, to an unknown depth, underlay the present (2012) area of the robust contiguous lemonade berry stand between the top of the coastal bluff and elevations 80 to 90 feet. Weathered colluvium and fractured or displaced and sheared Monterey Formation materials are exposed along most of the 40-foot high coastal bluff face. Rilling, variously partly covered by vegetation, occurs on the coastal bluff face from near the top to near the toe of the coastal bluff. Where visible, the coastal bluff face displays weathered and fractured Monterey Formation shale that is oriented nearly east-west with out-of-slope dips generally ranging from 10 to 25 degrees.

3.1.1. Aerial Photograph Analysis - We analyzed multiple sets of stereo-pair aerial photographs to evaluate the parcel and to document topographic changes between 1929 and 2005. Our evaluation of the 1929 aerial photos reveals that a pre-existing landslide was located in the general vicinity of the 1978 landslide. This landslide contained dormant landside debris to the east and west, but intact ground upslope. In 1929, a prominent linear drainage channel extended from farmland upslope (north), east of present day Oliver Road to the pre-existing landslide area and likely contributed to the slope instability. The prominent persistent coastal bluff is visible along the shoreline on the subject parcel, immediately to the north and upslope from the back beach. No coastal bluff "upper riser" occurs on the historical aerial photography of the area of the subject parcel.

We scanned the 1950 aerial photograph and overlayed it onto the 1962 photograph in ArcMap format to determine the location of the pre-existing landslide with respect to the existing residential structures, to the 1978 landslide, and to the location of the proposed structure. Based upon this geo-referencing exercise, it is apparent that the 1978 landslide occurred in virtually the same location as the pre-existing landslide limits, particularly in the headscarp area. The eastern portion of the 1978 landslide appeared to have incorporated the older landslide debris visible in the 1929 aerial photographs. No pre-existing landslide debris is located upslope of the proposed residence at 1925 ECDLL.

The 1938 and 1950 aerial photographs reveal similar features as the 1929 aerial photographs, and also show active shallow slumps and slides along the face of the coastal bluff. The 1962 aerial photographs show that most of the residential structures along El Camino De La Luz are in place, including the house on the subject parcel. It is apparent in 1962 stereo-paired photographs, that the houses at 1925 and 1921 El Camino De La Luz were placed atop the contact between intact rock and the pre-existing landslide debris. Prior to the 1978 landslide, substantial artificial fill was also placed on 1937 El Camino De La Luz to support a driveway, which extended downslope atop the pre-existing landslide debris.

The 1978 oblique aerial photographs obtained by Weaver (1978) depict a large, relatively intact landslide block that has pushed seaward up to approximately 80 feet. A prominent headscarp can be seen, exposing light colored Monterey Formation bedrock. The surf zone has been altered by the intrusion of the landslide debris, and wave action is deflecting around this debris. In the Pacific Materials Lab, Inc. (1978) report, they mentioned that the surf zone had been uplifted in front of the coastal bluff, and their interpretation was that a deep rotational slide had uplifted these materials. It is our interpretation that the landslide was translational, and the landslide toe was near the base of the coastal bluff, and that the landslide 'bulldozed' the cobble and beach sand upward, rather than by rotational lifting. By October 1979, the majority of the landslide debris protruding into the surf zone had been eroded, and the grading performed in the headscarp area was complete. In a 1987 aerial oblique, the grading performed in 1984 for the buttress at 1933 ECDLL is clearly visible. In this photo it is clear that a substantial cut was performed near the top of the coastal bluff and this material was placed up under the residence as fill. The coastal bluff is clearly visible in this photograph as a prominent, precipitous, denuded slope face with a very sharp break in slope at the top of the exposed bluff face.

3.1.2 1978 Landslide – On February 14, 1978, a landslide occurred along the portions of approximately 8 parcels, including at 1925 El Camino De La Luz (ECDLL). We understand that initial movement was noticed as early as February 11, 1978 as tension cracks in the headscarp region. As a result of the catastrophic landslide movements that occurred late in the evening of February, 14, 1978, the residence at this address was destroyed, along with the residence on the adjoining parcel at 1921 ECDLL. The residence at 1925 ECDLL was constructed of concrete blocks on a shallow foundation that straddled the landslide headscarp, which resulted in most of the concrete block foundation being translated downslope. The residence at 1933 ECDLL was not directly impacted, but was severely undermined, as were portions of the patio. An oblique low-altitude aerial photograph taken by or on behalf of Weaver shortly after the 1978 El Camino de la Luz landslide indicates a prominent lobe of landslide debris that extends from 1925-1927 ECDLL through 1909-1903 ECDLL in a south-southeasterly wedge onto the beach platform. Although parts of that wedge were subsequently eroded by marine processes, and excavated in 1984 along westerly 1925 ECDLL between el. 90 feet and elevation 50 feet as part of the Doolittle grading to buttress the 1978 landslide headscarp at 2001-1933-1927 ECDLL, the 1978 landslide appears to have been triggered by saturation of the oxidized Monterey Formation from upslope anthropogenic sources during the above-average 1977-1978 rain year, rather than by marine undercutting of the distinct coastal bluff.

In or about August-September, 1978, the City variously removed or buried landslide structural debris at 1925 ECDLL in conjunction with grading of the Mesa Trunk Line Sewer earthen buttress, to elevation 126 feet, and demolished the remaining standing part of the house at adjacent 1921 El Camino de la Luz. The City's grading extended between elevations 60 and 126 feet at 1925 El Camino de la Luz and was augmented by installation of sand bag walls, drainage flex pipe, a low berm near the head of the driveway, and v-ditch drainage to direct stormwater runoff to the relict drainage swale on the coastal bluff top at 1921 El Camino de la Luz. In 1984, grading (excavation) by Doolittle directed stormwater runoff from the westerly band of 1925 El Camino de la Luz, between elevations 90 and 55 feet, toward 1927 ECDLL.

Our analysis indicates that the 1978 landslide failed along, or near, the limits of the pre-existing landslide. However, we do not see evidence in the historical aerial photographs, or from our geologic mapping, for older landslide debris extending upslope of the 1978 landslide in the driveway segment of 1925 ECDLL, the street, or lots to the north. Based upon geologic relationships identified in the field and in aerial photographs, and our subsurface exploration, CSA interprets the 1978 landslide to be a translational rock-block landslide. The current (2011) exploration of this landslide reveals a basal landslide rupture surface depth of approximately 0 to 11 feet within the southern portion of the proposed building envelope of 1925 ECDLL, and a maximum depth of approximately 40 to 50 feet downslope near the coastal bluff (see Figure 6, Engineering Geologic Cross Section A-A'). Historical photographs reveal that the headscarp of the 1978 landslide, in the middle to eastern portion of the landslide (including the 1925 ECDLL parcel), has a relatively moderate inclination (i.e., less than 35 degrees), and likely failed along bedrock stratification inclined seaward (i.e., dip-slope bedding). Aerial photographs reveal fence line offsets that indicate the landslide moved slightly southeastward, which is slightly oblique to the bedding orientations documented in the headscarp region within large diameter boreholes, but near-parallel to the orientations near the beach. In our investigation, we documented southwesterly dips ranging from 25 to 35 degrees in the upper portion of the slope, but found much shallower bedrock inclinations (10 to 20 degrees) and more southerly orientations in the bluff face and in the surf zone.

The residence on 1925 ECDLL prior to the 1978 landslide was, in part, constructed atop the margin of the older landslide observed in the historical aerial photographs and mapping, thus was straddling the contact between landslide debris and intact rock. The lack of deep foundation support and poor placement of the structure resulted in its experiencing downslope movement when the landslide debris reactivated, removing support for the southern side of the structure. Unlike that location and foundation, the current residential project locates the proposed house exclusively within intact Monterey Formation bedrock within the headscarp of the evacuated landslide, on a deep pier and grade beam foundation and in an area bounded to the south (downslope) and north (upslope) by shear pin (stitch-pier) walls. As illustrated in Figures 5 and 6, Engineering Geologic Map and Engineering Geologic Cross Section A-A', the northern portion of the 1978 landslide debris is mostly removed from the area of the proposed building envelope, with only a small wedge located in the southern portion of the building envelope, which is proposed to be stabilized with a tied-back shear pin wall.

3.2 <u>Local Geology</u>

The parcel is underlain by bedrock materials of the Monterey Formation (i.e., well-stratified siltstone and shale with isolated interbedded bentonite horizons). Alluvial terrace materials and artificial fill are located beneath and east of the driveway in the northern "flag pole" portion of the 'flag lot' parcel. Oxidized bedrock materials on the parcel experienced landsliding in 1978. These displaced bedrock materials are well-exposed on the coastal bluff face on the parcel. Bedrock materials are exposed along the coastal bluff on the upcoast and downcoast sides of the 1978 ECDLL landslide. Bedrock orientations, in general, are documented to be very similar within the landslide mass as they are outside of the landslide, with northwest strikes and southwest dips ranging from 15 to 35 degrees. Bedrock materials are locally overlain by alluvial terrace deposits in the northern portion of the site, landslide deposits in the southern portion of the site, surficial soil materials, and artificial fill. The 1978 ECDLL landslide mobilized along the central and southern portions of the site and resulted in residential distress and the ultimate removal of the two residential structures (1921 and 1925 El Camino De La Luz). The City graded the area of the parcel between elevation 60 and 126 feet on or about August-September 1978 and installed sand bag walls, flex pipe, and v-ditch drainage to direct stormwater runoff to the relict drainage swale on the 1921 El Camino De La Luz coastal bluff top.

3.3 Earth Materials

Earth materials present in the subject parcel include beach deposits, landslide deposits, surficial soil/colluvium, non-marine terrace deposits and Monterey Formation bedrock materials as described below:

3.3.1 Beach Deposits (Qbs) - Beach deposits are present in the subject area seasonally as a thin veneer of well-sorted medium-grained sand atop bedrock materials and coarse cobble deposits. The beach deposits are derived from local and regional weathering of the coastline and net longshore drift currents that transport materials from west to east. The beach deposits are light to medium gray in color, medium grained, well sorted and rounded, contain abundant shell fragments, and contain occasional rounded cobbles up to 6 inches in dimension. A more persistent accumulation of very coarse cobbles is located at the landward edge of the beach and consists of very hard and strong, fossiliferous, dolomitic and siliceous cobbles derived from mass wasting of the local Monterey Formation.

3.3.2 Landslide Debris (Als) – Landslide debris associated with the 1978 landslide, as illustrated on our Engineering Geologic Map (see reduced version, Figure 5), underlies the southern portion of the parcel. Our geologic mapping, logging of small-diameter boreholes, and downhole logging of large-diameter borehole LD-3 reveals that the landslide deposits consist of highly fractured, dilated, disrupted, and weathered Monterey Formation sedimentary materials consisting of mudstone, siltstone, and shale. The landslide debris contains bedrock orientations near the regional conditions, consistent with a translational type of earth movement. The bedrock orientations on either side of the basal rupture surface in LD-3 are virtually identical; by definition, this necessitates a translational failure mechanism. The basal rupture surface is not exposed in the coastal bluff, and our interpretation suggests that the 1978 landslide ‘toed-out’ near the shoreline angle defined by the cobble deposits on the back beach. We did observe a shear surface near the top of the cobble deposits, but our interpretation is that this shear surface is likely an intermediate shear above the buried basal shear surface.

3.3.3 Non-Marine Terrace Deposits (Qt) – Non-marine terrace deposits unconformably overlie Monterey Formation bedrock across the driveway portion of the parcel, between elevations 130 and 140 feet. The non-marine terrace deposits range in thickness from approximately 5 to 8 feet. These materials are characterized by a dark brown, organic rich, silty clay soil profile on which vegetation is established. This soil profile grades down into medium brown silty sands with occasional pebbles up to 4 inches scattered throughout. These silty sands, which form the main body of the terrace materials, also contain laterally discontinuous lenses of well-rounded pebbles that represent channel gravel deposits. At the base of the terrace is a semi-continuous gravel lag deposit that is directly atop the siltstone bedrock. These earth materials were exposed and logged in a hand-excavated test pit just below the end of the driveway by Weaver.

3.3.4 Monterey Formation Bedrock (Tm) – The parcel is underlain by sedimentary bedrock materials of the Monterey Formation. These earth materials consist of variably weathered siliceous shale, mudstone, siltstone, and isolated fine sandstone. The Monterey Formation is generally thin-bedded and has a weathering zone of between 25 and 30 feet thickness consisting of tan/buff shale and mudstone that is closely fractured and weak to moderately strong. Isolated, deeply weathered mudstone interbeds were observed in the subsurface during downhole logging of LD-1 and LD-2. Below the weathering zone, the bedrock is dark brown to dark gray, petroliferous, and well-bedded, with isolated tightly-folded rock having numerous tar seams. Bedrock orientations observed in the large-diameter borings and in the coastal bluff reveal persistent east-west to northwest strikes and southerly dips of moderate inclination, resulting in dip-slope conditions (sedimentary bedrock layers sloping downslope) at the bluff face.

3.4 Subsurface Conditions

The subsurface geologic conditions at 1925 El Camino De La Luz were explored by excavating three small-diameter boreholes in May 2011, and two large-diameter boreholes in October 2011 (see Plate 1 and Figure 5, Engineering Geologic Map).

3.4.1 Small-Diameter Borehole Exploration – Three small-diameter boreholes were drilled on the 1925 El Camino De La Luz parcel, and two borings were drilled on the adjacent 1921 El Camino De La Luz parcel**.** The small-diameter boreholes were excavated primarily utilizing mud-rotary, continuous coring methods; however, the upper portions of the boreholes within artificial fill, weathered bedrock and landslide debris were drilled using hollow-stem auger techniques. Inclinometer casing was installed in each borehole, and 2 vibrating wire piezometers were attached to the outside of each borehole casing and grouted into place. Detailed logs of these borings were generated in the field and augmented by additional logging of retained cores in our lab by staff and principal geologists (see borehole logs in Appendix A).

Borehole B-1 – Borehole B-1 was excavated along the east side of the 1925 El Camino De La Luz driveway, near elevation 132 feet and upslope of the proposed building envelope. In Borehole B-1, we encountered approximately 7 feet thickness of non-bedrock material consisting of silty clay, silty sand terrace deposits, and possibly some artificial fill. Below these earth materials, we encountered weathered Monterey Formation bedrock consisting of weak to moderately strong, fractured, laminated shale and claystone. The terrace deposits and weathered bedrock in the upper 28 feet below ground surface (to elevation 104 feet) were drilled using a hollow-stem auger with Modified California sampling at approximate 3-foot intervals. Approaching the boundary between the weathered and unweathered bedrock, the drilling became much harder, and mud-rotary drilling methods were used with continuous core sampling (see Appendix A, representative photographs of core samples). We encountered slightly weathered to unweathered Monterey Formation bedrock from 38 feet depth (elevation 94 feet) to the bottom of the borehole at a depth of 98 feet below ground surface (elevation 34 feet). These earth materials consisted of unoxidized, variably fractured, and moderately strong to strong shale and claystone. Localized highly fractured and weak zones were encountered, and isolated clay seams were encountered at depths of 70 and 75 feet below ground surface. No bentonite layers were encountered, but numerous tar seams were encountered in Borehole B-1.

Borehole B-2 – Borehole B-2 was excavated within the proposed building envelope, near elevation 119 feet. In Borehole B-2, we encountered approximately 5 feet of non-bedrock material consisting of silty clay and silty sand artificial fill. Below these materials, weathered Monterey Formation bedrock was encountered, consisting of weak to moderately strong, fractured, laminated shale and claystone. The fill and weathered bedrock in the upper 30 feet were drilled using a hollow-stem auger with Modified California sampling at approximate 3- to 5-foot intervals. Approaching the boundary between the weathered and unweathered bedrock, the drilling became much harder, and mud-rotary drilling methods were used with continuous core sampling (see Appendix A, representative photographs of core samples). We encountered slightly weathered to unweathered Monterey Formation bedrock from 35 feet to the bottom of the borehole at a depth of 98 feet below ground surface. These materials consisted of unoxidized, variably fractured, and moderately strong to strong shale and claystone. Localized highly fractured and weak zones were encountered, and isolated, unsheared clay seams and/or thin (paper thin to a few mm thick) clay films were encountered at depths of 31, 89 and 95 feet below ground surface (elevations 101, 43, and 37 feet). No bentonite layers, but numerous tar seams were encountered in Borehole B-2.

Borehole B-3 – Borehole B-3 was excavated downslope of the proposed building envelope, near elevation 87 feet. In Borehole B-3, we encountered approximately 14 feet thickness of landslide deposits consisting of silty clay with rock fragments, disrupted shale and claystone with numerous interstices (voids). Below these earth materials, weathered Monterey Formation bedrock was encountered, consisting of moderately strong, fractured, laminated shale and claystone. The fill and landslide debris in the upper 10 feet were drilled using a hollow-stem auger with Modified California sampling at approximate 3-foot intervals. Mud-rotary drilling methods were used below a depth of 10 feet, with continuous core sampling. We encountered slightly weathered to unweathered Monterey Formation bedrock from 18 feet depth to the bottom of the borehole at a depth of 98 feet below ground surface. These materials consisted of unoxidized, variably fractured, and moderately strong to strong shale and claystone. Localized highly fractured and weak zones were encountered, but no bentonite layers were encountered. Numerous tar seams were encountered throughout the relatively unweathered portion of the borehole and these seams typically cross-cut bedding along very tight fracture zones.

3.4.2 **Instrumentation** – Inclinometers and piezometers were installed within boreholes B-1, B-2, and B-3. Inclinometers were installed the full length of each borehole

(98 feet depth below ground surface) and initialized on May 19, 2011. Subsequent inclinometer readings were performed on May 26, June 1, June 27, and September 22, 2011, and January 5, 2012. To date, we have not detected movements in any of the three inclinometers that exceed the precision limitations of the instrument (see Appendix C, inclinometer graphs). Two, vibrating wire piezometer sensors were installed in each of the three boreholes at mid-range depths of between 38 and 45 feet, and deep sensors between 88 and 93 feet depth below ground surface. The water levels recorded in these instruments (see Appendix C, piezometer graphs) between mid-May 2011 and early January, 2012 have been relatively consistent and generally correspond with the transition from oxidized to unoxidized bedrock between 25 and 30 feet depth below the ground surface, as shown on Engineering Geologic Cross Section A-A' (see Figure 6).

3.4.3 Large-Diameter Borehole Exploration – We excavated two, 24-inch diameter, large-diameter boreholes (LD-2 and LD-3) for the purpose of investigating the subsurface conditions of 1925 El Camino De La Luz. A third large diameter borehole (LD-1) was excavated on the adjacent property at 1921 ECDLL. These boreholes were downhole logged by our staff and principal geologists; our geologic logs are included in Appendix A. Figure 5 also shows the location of these boreholes.

Borehole LD-2 – Borehole LD-2 was excavated within the proposed building envelope of 1925 El Camino De La Luz, at an approximate ground surface elevation of 120 feet, upslope of the 1978 landslide debris, and within the headscarp region of the 1978 landslide. In Borehole LD-2, we encountered approximately 5 feet thickness of artificial fill in the southern (downslope) portion of the borehole only, which we interpret to be part of the post-1978 landslide grading. This fill was not observed in the northern (upslope) portion of the hole. The entire borehole (with the exception of the artificial fill) consists of relatively uniform Monterey Formation bedrock characterized by: deeply to moderately weathered, thin-bedded claystone and shale, weak to moderately strong, closely fractured, dry to moist in the upper 15 feet, and moist to locally wet below 15 feet. Conspicuous tar seams were observed cross-cutting bedding. The claystone interbeds were locally soft and plastic, whereas the shale was generally moderately hard, strong, and brittle. Bedding orientations were generally consistent, with average orientations documented to be N63W, 27SW. The borehole did not contain evidence of previous landsliding, such as open fractures, shears, offset bedding relationships, striated shear planes or abundant high-angle fractures with roots and openings. Very difficult drilling conditions were encountered at a depth of 26 feet. Seepage was encountered at a depth of 23 feet during drilling, and standing water stabilized in the borehole at a depth of 21.5 feet at the end of the day. Photographs and video footage of this borehole are on file at CSA's Los Gatos office.

Borehole LD-3 – Borehole LD-3 is located downslope of LD-2 within the trailing edge of the 1978 ECDLL landslide debris, at approximate elevation 93 feet near the southeastern margin of the proposed building envelope. This borehole was excavated near the north-south property line, and several feet onto the 1921 El Camino De La Luz parcel. In Borehole LD-3, we encountered approximately 12 feet thickness of landslide debris characterized by profusely fractured, unstable, ravelly, deeply weathered and weak Monterey Formation materials. At a depth of 12 feet, we encountered a prominent basal landslide shear surface characterized by a 3- to 5-inch thick shear zone with upper and lower bounding shear surfaces that are bedding plane-parallel. Between the upper and lower bounding shears, we documented highly fractured, disrupted, and displaced silty sand and clayey silt with abundant shale rock fragments. The shear zone has an orientation of N70W, 30SW, which is similar to bedding orientations above and below. Clay gouge, paper thin to 0.25-inch in thickness, was observed along the upper and lower contacts of the shear zone, but these gouge zones were discontinuous and disrupted. We interpret this to be the result of dilation (unloading) along the shear zone as the main portion of the landslide evacuated this area and the trailing edge of the 1978 landslide came to rest at this location.

The basal shear zone defines a dramatic textural change in the borehole, where unoxidized, intact, hard and strong Monterey Formation bedrock materials were encountered below the shear, and extended to the bottom of the borehole at a depth of 30 feet. Bedding orientations were generally consistent above and below the landslide shear surface, with average orientations documented to be N67W, 28SW. Practical drilling refusal was encountered at a depth of 30 feet. Seepage was encountered during drilling at a depth of 25 feet, and standing water stabilized in the borehole at a depth of 26 feet the following morning. Photographs and video footage of this borehole are on file at CSA's Los Gatos office.

Borehole LD-1 – This borehole was drilled in the northern portion of 1921 El Camino De La Luz, outside of the limits of the 1978 landslide, and was used for correlation purposes in the analysis of the subsurface geology on 1925 El Camino De La Luz. In borehole LD-3, we encountered relatively uniform Monterey Formation bedrock for the entire depth of the hole (25.4 feet), characterized by: deeply to moderately weathered, thin-bedded claystone and shale, weak to moderately strong, closely fractured, dry to damp for the entire borehole, and conspicuous tar seams along bedding near 18 feet. The claystone interbeds were locally soft and plastic, whereas the shale was moderately hard, strong, and brittle. Bedding orientations were generally consistent, with average orientations documented to be N40W, 33SW. The borehole did not contain evidence of previous landsliding, such as open fractures, shears, offset bedding relationships, striated bedding planes or abundant high-angle fractures with roots and openings. Very difficult drilling conditions were encountered at a depth of 25 feet. Seepage was not encountered in this borehole. Photographs and video footage of this borehole are on file at CSA's Los Gatos office.

3.5 Groundwater Conditions

Groundwater was observed in large-diameter boreholes LD-2 and LD-3 at stabilized (approximate 12-hour period) depths of 21.5 and 26 feet, respectively. Relatively minor seepage was observed during drilling and logging of these holes at depths of 23 feet (LD-2), and 25 feet (LD-3). Rotary wash drilling methods were utilized for the small-diameter drilling, and thus, no groundwater levels could be observed during drilling. However, two vibrating wire piezometer sensors were installed in each of the three small-diameter boreholes; one at a mid-range depth of between 38 and 45 feet, and one deep sensor between 88 and 93 feet below ground surface. Water levels were recorded in these instruments (see piezometer graphs, Appendix C) between mid-May 2011 and early January 2012. Groundwater levels have been relatively consistent during the monitoring period, and generally correspond with the transition from oxidized to unoxidized bedrock between 22 and 35 feet below the ground surface, as shown on Engineering Geologic Cross Section A-A' (see Figure 6). The deep and shallow sensors reveal groundwater levels nearly at the same elevation, indicating that neither artesian conditions nor perched water tables are present on the parcel.

3.6 Summary of Findings

Our surface and subsurface exploration reveals that the parcel at 1925 ECDLL is underlain by sedimentary bedrock materials of the Monterey Formation. These materials are generally oriented in a dip-slope condition with bedding sloped approximately 20 to 35 degrees to the south. The intact Monterey Formation at the subject parcel contains a prominent weathered zone that is approximately 20 to 40 feet in thickness. The central and southern portions of 1925 ECDLL are underalin by a moderately deep (i.e., 40 to 50 feet in maximum depth), translational landslide. This landslide mobilized in February, 1978 in response to heavy rainfall and likely in response to stormwater discharge from ECDLL to the south trending driveways, and leakage from the Mesa Trunk Sewer Line into the headscarp area of the landslide. This landslide debris is in a low slope position on 1925 ECDLL and has not shown measurable reactivation since the primary failure in 1978; however, it is our opinion that reactivation of the debris could occur during heavy rainfall or seismic events. No evidence of pre-historic landsliding was observed upslope of the 1978 landslide. The mobilization of the 1978 landslide mass removed lateral subjacent support for the northern portion of 1925 ECDLL, and thus, thorough characterization and analysis of this slope has been performed to determine its stability state, as describe later in this report.

4.0 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSES

4.1 Representative Cross Section Analyzed

Engineering Geologic Cross Section A-A' (see Figure 6 and Plate 1) was analyzed for our slope stability analyses because it best represents the site topography and subsurface exploratory borehole data. This cross section was generated based upon our site-specific topographic survey of 2010 as well as our small- and large-diameter subsurface exploration and monitoring of groundwater levels.

4.2 Analysis Procedure

Our analysis was focused on evaluating the stability of the slopes at the parcel with the conservative assumption that the 1978 landslide mass below the proposed building envelope could potentially/eventually evacuate/mobilize seaward, and be subsequently eroded away by nearshore coastal processes. In reality, the remaining terrestrial landslide mass has not moved significantly since 1978, and is therefore likely actually providing some buttressing effect for the materials upslope. For conservatism, we did not rely on this buttressing component in our analysis and assumed that the existing landslide debris was no longer present. In our analysis, we evaluated the resulting slope stability by determining the required stabilization structures and shear resistance demand needed to mitigate the potential for future deep-seated slope instability to industry standard factors of safety. We evaluated the following four conditions:

1. We evaluated the future slope stability of the parcel by assuming that the landslide debris had evacuated with no improvements (residences) or stabilization made to the parcel;

2. We evaluated the future slope stability of the parcel by assuming that the landslide debris has evacuated, the residence has been constructed and the slope is stabilized with two rows of shear pins, the lower row equipped with tiebacks to achieve a satisfactory static Factor of Safety (FS) equal to or greater than 1.5;

3. We evaluated the same condition as 2, above, except that a pseudo-static coefficient of k=0.15 is added to model potential earthquake loading and a

satisfactory pseudo-static Factor of Safety (FS) equal to or greater than 1.1 to 1.2 is achieved; and

4.3 Software and Methodology

The computer program SLOPE/W (by Geo-Slope International) was used to perform the CSA slope stability analysis. All analyses were performed utilizing the Morgenstern-Price method of analysis. With this method, the parameters of our representative cross section are input into the program, the potential sliding mass is divided into slices by the program, and inter-slice side force inclinations are variable. The Morgenstern-Price method satisfies equilibrium conditions for overall moment, individual slice moment, and vertical and horizontal forces.

The Block Search method was used to identify hypothetical critical potential failure surfaces. This method is preferred for analyzing non-circular surfaces along bedding planes. Using the Block Search method, trial failure surfaces are created by connecting surfaces (linear lines) between nodes of user-specified blocks. The number of nodes and sizes/locations of blocks are parameters selected by the program operator. In addition, the operator can specify a range of projection angles of each block to the ground surface at both the head of the slide and the toe of the slide. This method of searching creates a large number of surfaces and significant iteration is required.

4.4 Analysis Input Parameters

The primary input parameters needed for the limit equilibrium sliding stability analyses included the ground surface topography, subsurface geologic stratigraphy, depth and inclination (geometry) of the hypothetical basal shear surface, strength and unit weight properties of the earth materials and the groundwater levels.

Ground Surface Topography – The topography was based on the site-specific 2010 CSA topographic survey (with 2-foot contour intervals) of the parcel.

Subsurface Geologic Stratigraphy – The geologic units were determined based on surface mapping and subsurface exploration, including small-diameter borings and downhole logging of large-diameter borings. The stratigraphy is comprised of two Monterey Formation bedrock units (Weathered Bedrock and Unweathered Bedrock), a Quaternary Terrace (Qt) deposit, Artificial Fill (Af), and Landslide Debris (Als).

Shear Strengths - We performed back-calculation analysis on the existing landslide and, based on that analysis, arrived at a residual shear strength parameters and average wet unit weight of C= 0 psf, Ø = 19^0, γ = 89 pcf for this material. The Weathered and Unweathered Bedrock were assigned along-bedding shear strength parameters and wet unit weight values of C = 0 psf, Ø = 25^0, γ = 105 pcf and C = 1 psf, Ø = 32^0, γ = 110 pcf, respectively. The Weathered Bedrock shear strength parameters of C = 0 psf, Ø = 25^0 were based on an average of four residual torsional ring shear test results from weathered bedrock samples taken by CSA staff geologists in the large-diameter boreholes. The Unweathered Bedrock shear strength parameters of C = 1 psf, Ø = 32^0 represent the highest residual torsional ring shear test result of the four weathered bedrock samples. The wet unit weight value selected for the Weathered Bedrock represents the average of seven unit weight tests, while the Unweathered Bedrock wet unit weight was selected based on an average of the five highest laboratory test result values. We note that the unweathered, along-bedding shear strength values were approximately 28% greater than the weathered shear strength values.

Cross-Bedding Shear Strength - During our geologic mapping, logging of small-diameter continuous cores and downhole logging of the large-diameter borings within the intact Monterey Formation bedrock, we observed layered stratigraphic rock with unit strengths that varied from very hard and strong rock, to weak to moderately strong rock. This layered, intact rock inherently contains significant cross-bedding shear strength in both the weathered and unweathered bedrock relative to along bedding shear strength. For the slope stability analysis, we used a cross-bedding shear strength in the weathered material based on the results of two of the four triaxial compression consolidated shear strength tests performed which resulted in anistropic shear strength parameters of C = 0 psf, Ø = 45^0.

Since difficult drilling conditions were encountered in the unweathered bedrock, we could not obtain undisturbed tube samples; therefore, mud-rotary drilling was performed and continuous core samples were retrieved. The shear strength parameters (friction angle and cohesion) for the unweathered rock were consequently derived by using an estimate of the rock mass strength of the sedimentary rock using the Hoek-Brown failure criterion. This procedure consists of an empirical method used to estimate the strength parameters of jointed rock masses (Hoek and Brown, 1997). Input parameters to this analysis include the Geological Strength Index (GSI, Wyllie and Mah, 2004; and Rocscience, 2007), which is a rating system that allows the rock mass quality to be quantified based on the surface conditions and rock fracturing. Other input parameters include unconfined compressive strength and rock type rating (a factor, m, which is a constant dependent on rock type) related to the principal stresses at failure.

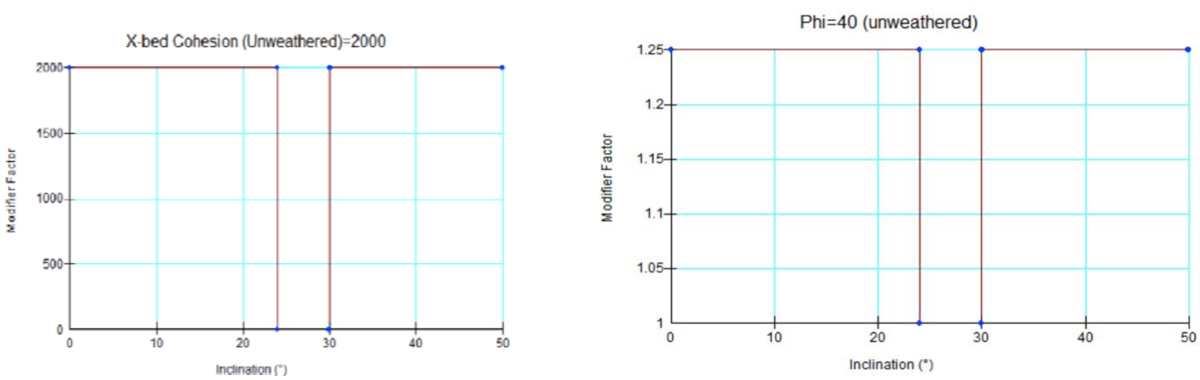
We logged the core samples in detail and found that the unweathered Monterey Formation consists of three distinct rock domains based upon strength characteristics: 1) Concretionary Siltstone, very hard and very strong, limited to approximately 5% of the unweathered stratigraphy, GSI 55-70; 2) Siliceous Siltstone, medium strength, well-bedded, closely fractured, makes up approximately 65% of the unweathered stratigraphy, GSI 50-55; and 3) Weakly Cemented Siltstone, very weak to weak strength, well-bedded, makes up approximately 30% of the unweathered stratigraphy, GSI 20-25. We used a Schmidt Hammer to obtain the compressive strength of the rock, and correlated these test results with geologic field tests (hammer blows, knife scratch tests, etc.) to assure appropriate compressive strength input parameters were being used. The results of the calculations (performed using Rocscience, 1997 software) are presented in Appendix D.

In summary, the Hoek-Brown failure criterion (Mohr Coulomb cohesion strength parameter) resulted in Phi = 40 degrees and cohesion of 3,700 psf. It should be noted that for conservatism, we utilized exclusively the lower bound range of the GSI input parameters for the Case 1 and Case 2 rock domains. For Case 3, the upper bound input parameters (GSI = 25 and 0.5 ksi compressive strength) were used rather than the lower bound (GSI = 20 and 0.15 ksi compressive strength), due to the low strength (Phi= 14°, C = 720 psf) of the results, which are unlikely to be achieved for these intact, unweathered bedrock materials. A weighted average of the three rock domains, and their associated strengths, was then generated to arrive at average shear strength parameters for the unweathered rock that would have to be sheared in order for a landslide shear plane to daylight at the toe of slope. This weighted average (Phi = 40°, C = 3,700 psf) was reduced to Phi = 40°, C = 2,000 psf to conservatively account for uncertainties.

It is worth noting that cross-bedding strengths for the Monterey Formation at the Ocean Trails Development project on the Palos Verdes Peninsula (with similar Monterey Formation bedrock type and similar stratigraphy as the subject parcel) were derived by the Project Geologist in much the same manner, except they used the Rock Mass Rating system. They arrived at a cross-bedded shear strength of Phi = 51° and C = 6,500 psf. These data were peer reviewed by an independent panel of experts who determined that a range of strengths was more appropriate at the site, depending upon anticipated loading conditions. The lower bounding strength parameters were Phi = 40° and C = 2,000 psf, with the upper bound strength parameters at Phi = 51° and C = 6,500 psf. As peer reviewers for the project, we inspected hundreds of feet of core and downhole logged over 50 large-diameter boreholes at the Palos Verdes site, and conclude that the rock at the subject West Mesa parcel is very similar to that observed at Ocean Trails. Therefore, it is our opinion that the cross-bedding strengths for the unweathered bedrock presented above are conservative, and are generally used for rock that is more disrupted than that observed in the unweathered zone at the subject West Mesa parcel.

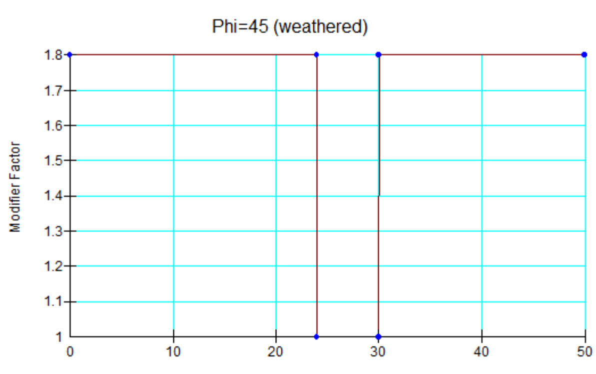
To model this anisotropic strength in the computer program, we specified the along-bedding shear strength as failure surface inclinations between 24 and 30 degrees (see Figure 9) and any shear outside of this range would be assigned a cross bedding factor as discussed above and shown in the following Table 2.

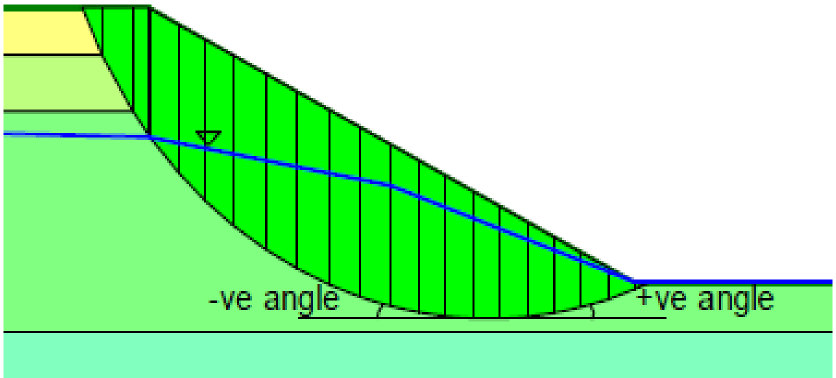
Figure 9. Anistropic Strength Function for Bedding Inclination



The inclination angles measured from the positive x-axis are positive, and negative when measured from the negative x-axis (see Figure 10 below).

Figure 10. Sign Convention for Anisotropic Strength Function





(Source: Geostudio 2007 SLOPE/W)

Table 2 – Slope Stability Earth Material Properties

Earth Material	Unit Weight (pcf)	Along Bedding Cohesion (psf)	Along Bedding Friction Angle (degrees)	Cross Bedding Cohesion (psf)	Cross Bedding Friction Angle (degrees)
Weathered Bedrock	105	0	25	0	45
Unweathered Bedrock	110	1	32	2,000	40

For simplicity, the relatively thin zones of Artificial fill (Af) and Quaternary Terrace (Qt) materials were modeled as Weathered Bedrock.

Groundwater Levels – A piezometric surface was selected using the highest recorded water levels indicated by the piezometers installed in the small-diameter boreholes. From the furthest downslope piezometer, the water surface was assumed to gradually descend towards, and daylight at the toe of, the slope just above the back beach.

Building Load – We modeled the proposed residential development loads assuming two parallel continuous point loads of 850 lbs, 20 feet apart, for the proposed residential foundation piers.

4.5 Analysis Results

Back-Calculation - We began our slope stability analysis by performing a back-calculation to determine shear strengths for forward analyses by assuming the existing area of the slope comprised of the 1978 landslide debris has a factor of safety slightly above unity (FS = 1.01), since the 1978 landslide is not presently moving. Consequently, for development of areas of the site within the active landslide boundaries, our goal was to raise the factor of safety from the current FS = 1.01 to industry accepted factors of safety of FS ≥ 1.5 static and FS ≥ 1.1 to 1.2 seismic (pseudo-static with a seismic coefficient of 0.15). The back-calculated condition represents Run 1 in the following Table 3. In this table, SSA stands for Slope Stability Analysis, SP stands for Shear Pin shear capacity needed, TB stands for Tieback pullout resistance capacity needed, k stands for seismic coefficient and FS stands for Factor of Safety.

TABLE 3 – SLOPE STABILITY ANALYSIS RESULTS

SSA Run #	Slope Condition Analyzed	Lower SP Wall (kips)	Upper SP Wall (kips)	TB (kips)	FS
1	Back-Calculation of existing slide debris surface	N/A	N/A	N/A	1.01
2	Slide debris In place (Critical Surface obtained from post-construction condition Run 5)	N/A	N/A	N/A	2.38
2A	Potential bedding failure with slide debris in place	N/A	N/A	N/A	2.41
3	Same as Run 2, but with slide debris removed (Critical Surface obtained from post-construction condition Run 5)	N/A	N/A	N/A	1.66
3A	Potential bedding failure with slide debris removed (same Critical Surface as Run 2A)	N/A	N/A	N/A	1.96
3B	Deep Block Search with debris removed	N/A	N/A	N/A	1.58
4	Shallow Surface from Run 7, no SP or slide debris	N/A	N/A	N/A	1.39
4A	Same as 4, but with slide debris in place	N/A	N/A	N/A	1.88
5	Critical Deep Surface, post construction of SP walls/ TBs / house loads, slide debris removed downslope of lower SP	40	50	100	1.68
6	Same Critical Surface and construction as Run 5, Seismic with k=0.15	40	50	100	1.23
7	Potential failure through upper shear pin wall (post construction, slide debris removed downslope of lower SP)	40	50	100	2.15
8	Same as Run 7, now with seismic k=0.15	40	50	100	1.24
9	Same as Run 5, with landslide debris in place downslope of lower SP	40	50	100	2.42
10	Same as Run 5, with landslide debris in place downslope of lower SP, now with seismic k=0.15	40	50	100	1.66

Forward Analyses - With the landslide debris evacuated, we conducted a slope stability analysis to determine what the most likely hypothetical deep failure surface (that deep basal shear surface geometry with the lowest factor of safety) would be with the upper portion of the parcel developed and stabilized (Run 5, FS = 1.68, in the table). Run 2 (FS = 2.38) represents an analysis of the factor of safety of this fully developed weakest potential shear surface if the landslide debris were still in place (not evacuated) and Run 2A (FS = 2.41) represents a search for the lowest factor of safety for a potential bedding plane failure with the landslide debris in place.

Also using the most like hypothetical failure surface from Run 5, Run 3 (FS = 1.66) represents what the factor of safety for this surface drops to if the landslide debris were to be fully evacuated, Run 3A (FS = 1.96) represents what the potential bedding plane failure factor of safety determined in Run 2A drops to if the landslide were to fully evacuate, and Run 3B (FS = 1.58) represents an analysis of the potential for a deep-seated landslide to develop with the landslide debris evacuated using a deep block search.

With the landslide debris evacuated, we also conducted a slope stability analysis to determine what the most likely shallow upslope hypothetical failure surface (that shallow basal shear surface geometry with the lowest factor of safety) would be with the upper portion of the parcel developed and stabilized (Run 7, FS = 2.15, in the table). Run 4 (FS = 1.39) represents an analysis of what the factor of safety for this shallow surface drops to if the landslide debris has evacuated and there are no shear pin walls constructed. Run 4A (FS = 1.88) represents the same analysis as Run 4, but with the landslide debris still in place.

As discussed above, Run 5 (FS = 1.68) represents an analysis of the most critical deep-seated failure potential of the slope if the parcel is developed with two shear pin walls (the lowermost of which also has tiebacks) and the landslide debris has been fully evacuated. Run 6 (FS = 1.23) is the pseudo-static analysis factor of safety of Run 5 using a seismic coefficient of k = 0.15.

As also discussed above, Run 7 (FS = 2.15) represents an analysis of the most critical shallow upslope (in the vicinity of the upper shear pin wall) failure potential of the slope if the parcel is developed with two shear pin walls (the lowermost of which also has tiebacks) and the landslide debris has been fully evacuated. Run 8 (FS = 1.24) is the pseudo-static analysis factor of safety of Run 7 using a seismic coefficient of k = 0.15.

Run 9 (FS = 2.42) is the same analysis as Run 5 (deep-seated stability), but with the landslide debris still in place as it is today and Run 10 (FS = 1.66) is the same analysis as Run 6, but with the landslide debris still in place as it is today. These runs should represent the static (FS = 2.42) and seismic (FS = 1.66) factors of safety of the developed portion of the parcel immediately after development. Should the existing remaining portion of the 1978 landslide reactivate and move downslope in the future, this deep-seated factor of safety for the developed portion of the property will decline, but should not decline below the levels of Run 5 (static FS = 1.68) and Run 6 (seismic FS = 1.23).

Worst Case Condition – We performed an analysis of what could be considered the worst case condition for slope instability of the upslope portion of the property by conservatively assuming that the 1978 landslide mass fully reactivated, mobilized and evacuated from the site to be eroded by coastal processes, and thus would no longer provide a buttress for the remaining slope to its landward side. This analysis indicated that, if the landslide debris evacuates, the remaining slope would have a reduced FS. Depending on the hypothetical failure surface analyzed, the FS could be reduced by as much as 30%. To relate this analysis to the proposed project, we chose the most critical hypothetical failure surface determined for deep-seated slope stability of the developed portion of the property with the landslide mass completely gone as discussed above with results listed in Table 3.

Shear Pin Walls with Tiebacks Installed for Stabilization – We performed numerous analysis iterations in which we varied the number of shear pin rows, shear pin resistance, number of tieback rows, and tieback tensions until the static factor of safety was above 1.5 for all hypothetically searched (both shallow and deep) potential failure surfaces. These analyses culminated in the design concept analyzed as described above with results listed in Table 3.

Shear Pin with Tiebacks Installed for Stabilization and a Seismic Coefficient – We fixed (held in place) the hypothetical critical surfaces found under the static loading conditions, and performed psuedo-static analyses using a seismic coefficient of k = 0.15. We again modified the shear pin resistance and tieback tensions until we achieved a seismic (pseudo-static) factor of safety greater than or equal to 1.1 to 1.2. These analyses culminated in the design concept analyzed as described above with results listed in Table 3.

Required Shear Pin Capacity – Our analysis indicates that satisfactory factors of safety (FS ≥ 1.5 static and FS ≥ 1.1 to 1.2 pseudo-static) can be achieved at 1925 ECDLL by installing two rows of 40-foot deep shear pins (steel-reinforced cast-in-drilled-hole piers) with an allowable (service) shear capacity of 40 kips for the lower row and 50 kips for the upper row, and spaced at 6-foot on-centers. The upper row would be installed roughly 40 feet south of the east-west property line between 1925 and 1927 El Camino

De La Luz (near grade elevations 113 to 116 feet), while the lower row of shear pins would be located approximately 77 feet further south of the upper row, or approximately 117 feet south of that east-west property line (near grade elevations 90 feet).

<u>Required Tieback Capacity</u> – Our analysis indicates that satisfactory factors of safety (FS ≥ 1.5 static and FS ≥ 1.1 to 1.2 pseudo-static) can be achieved by equipping the lower shear pins with one row of 65-foot long, 100-kip (design load) tensioned tiebacks declined at 20 degrees from horizontal upslope and located 7 feet down from the top of the shear pin.

Should the 1978 landslide mass reactivate, the lower shear pin wall has been designed to protect the building envelope from adverse slope stability impacts. Additionally, it is our opinion that there will be a sufficient lateral improvement in slope stability beyond the upcoast (west) and downcoast (east) edges of the shear pin wall to shield the subject parcel from adverse slope stability impacts associated with such an event.

<u>Horizontal Drains</u> – We recommend that horizontal drains, with clean-outs, be installed along the lower shear pin wall. The drains should be spaced approximately 18 feet apart, be inclined 2-degrees upslope, and extend a minimum of 100 feet into the slope in roughly a north-south direction, or perpendicular to the shear pin wall. Cleanouts should be installed at the collar of each drain, and each drain should feed into a collector pipe that discharges into a collection basin for reuse in residential landscaping, or into an appropriate storm drain pipe.

5.0 CONCLUSIONS REGARDING GEOLOGIC CONSTRAINTS AND GEOTECHNICAL FEASIBILITY OF RESIDENTIAL CONSTRUCTION

Based on our engineering geologic and geotechnical engineering investigation, it is our opinion that the northern portion of the 50-foot wide flag subarea on the flag lot parcel at 1925 El Camino De L Luz is suitable for proposed residential development provided that the recommendations of this report are incorporated into the design, construction, and operation of the project during its 75-year economic life. The proposed conceptual residential re-use building envelope, between (and within) elevations +90 and +130 feet MLLW, at 1925 El Camino De La Luz, Santa Barbara, California would be located upslope of the majority of the 1978 landslide debris and mostly within the relatively stable bedrock materials along the headscarp of the 1978 landslide. The proposed residential development envelope is located a minimum of 150 feet from (north of) the top of the coastal bluff on the subject parcel, and thus is and will be set back sufficiently from it during the 75-year economic life of the development in order to meet all applicable adopted setback standards, assure stability and structural integrity, minimize risks to life and property, and neither create, nor contribute significantly to, erosion, geologic instability, or destruction of the site or surrounding area. The proposed residential structure does not require the construction of protective devices that would substantially alter any natural bluff or cliff landforms, and none are proposed in this report or the coastal hazards/wave runup report by GeoSoils, Inc. for the subject parcel.

The City, in or about August-September, 1978, graded the 1978 ECDLL landslide on the subject parcel between elevations 60 and 130 feet, during which it partially buried the 1978 landslide headscarp with a shallow layer of artificial fill that constitutes non-engineered fill for foundation design recommendation purposes. We interpret the 1978 ECDLL landslide to have been a translational rockslide that failed along a pre-existing (i.e., pre-historical aerial imagery) landslide shear surface associated with an older landslide observed in the historical aerial photographs. There is no evidence in the subsurface or in the aerial photographs for an old landslide upslope (north) of the 1978 ECDLL landslide. From a geologic and geotechnical perspective, it is our opinion that a residential structure can be safely constructed within the proposed conceptual building envelope on the property, provided that the recommended deep foundation and subgrade landform stabilization shear pins, tiebacks and drains are incorporated into the residential reuse project plans for 1925 El Camino De La Luz.

We have generated Engineering Geologic Cross Section A-A′ through the central portion of the subject parcel to illustrate our interpretation of the geologic conditions and landslide geometry based on the results of our surface and subsurface exploration. The intent of the residential re-use building plan would be to construct the residence, and appurtenant structures, on the upper 50-foot wide flag portion of the flag lot parcel, upslope from most of the 1978 landslide debris, and with deep stabilization elements [i.e., shear pins (steel reinforced concrete caissons) with tiebacks for the lower row of pins]. This construction would likely include some minor temporary grading (estimated at approximately 200 cubic yards) necessary for pier and grade beam footings and tieback installation. The stability of the hillside upslope from the stabilization elements would be improved to the industry standard static and pseudo-static Safety Factors. Drainage control measures, both surface and subsurface, would be implemented at the parcel, including through interception of water for beneficial on-site reuse. In addition, the residential construction of the parcel should provide slope stability and drainage benefits to nearby residences, utilities, and the driveway upslope by increasing the factor of safety of the upslope area to minimum FS ≥ 1.5, and by preventing uncontrolled surface runoff from adversely impacting the existing landslide. Additionally, the horizontal drains should help reduce the risk of groundwater rise in the upslope area. Landform stabilization and residential reuse can occur at the subject parcel independently from adjacent parcels within the 1978 ECDLL landslide, all of which are under separate legal ownership from one another. CSA does not consider a seawall nor other shoreline protective structures to be necessary or required for the residential re-use of 1925 El Camino De La Luz, and none are recommended.

5.1 Potential Geologic Hazards

In our opinion, the potential primary engineering geologic and geotechnical constraints to residential reuse of the parcel at 1925 ECDLL are: 1) the potential for future instability in the oxidized Monterey Formation upslope of the 1978 landslide; 2) the potential for reactivation of the 1978 landslide in the middle/southern portion of the portion of the parcel, upslope from the top of coastal bluff; 3) the potential for translational landsliding incorporating both the upslope oxidized bedrock and unoxidized bedrock in the lower portion of the slope; 4) very strong seismic ground shaking associated with an earthquake along one of several nearby active faults; and 4) saturation of expansive earth materials. A detailed discussion of the geologic hazards and their potential to impact the site **follows:**

5.1.1 Landsliding - The primary engineering geologic constraint at this site is the 'dip slope' bedrock condition, which provides the potential for shallow to moderately deep landsliding to occur in bedrock along weak bedding planes. This dip slope condition results in the northern portion of the site having a safety factor, at present (2012), of less than 1.5. It should be noted that this condition extends upslope to include the City's Mesa Trunk Line Sewer, the driveway, and likely other residential properties located on shallow foundations along the headscarp of the 1978 landslide. For residential development of the parcel, mechanical, drainage and/or grading improvements are required to achieve a Safety Factor equal to or greater than 1.5. These improvements, while necessary to assure a stable building site, would also provide substantial stability benefits for the adjacent residentially developed upslope parcels, utilities, and infrastructure. For all of the above reasons and analyses, it is our opinion that the proposed development concept is consistent with the requirements of the State-certified City LCP, that new development shall both minimize risks to life and property in areas of high geologic, flood, or fire hazard, and assure stability and structural integrity, and neither create nor contribute significantly to erosion, geologic instability, or destruction of the site or surrounding area or in any way require the construction of protective devices that would substantially alter natural landforms along bluffs and cliffs.

By improving the drainage conditions and increasing the factor of safety of the hillside at the proposed residential development, as well as contributing to enhanced upslope and downslope landform stability, risks to life and property would be reduced (not only for this property but adjacent properties as well), stability and structural integrity would be assured, and the proposed development would in no way contribute to erosion or geologic instability. The placement of the residential structure in the northern portion of the 50-foot wide parcel would not constitute the stabilization of a natural landform along a bluff since this area was previously graded by the City in 1978 and in part under a coastal permit issued by the Coastal Commission in 1984, and the coastal bluff is located over 150 feet downslope of the proposed building envelope.

5.1.2 Reactivation of the 1978 Landslide – Reactivation of the 1978 landslide in some form is likely in the future, just as the 1978 ECDLL landslide was a reactivation of a previous landslide at this location. However, the present landslide debris associated with the 1978 landslide is in a relatively low slope position relative to the landslide headscarp. The proposed building envelope on the subject parcel is located largely upslope of this landslide debris and should not be impacted should the 1978 landslide debris reactivate (in contrast to the 1978 landslide condition where the residences straddled the contact between intact rock and landslide debris). Additionally, geotechnical recommendations for residential improvements upslope of the landslide debris have been generated assuming, conservatively, that the 1978 landslide debris has completely evacuated and is thus incapable of providing any measure of buttressing stabilization for the upslope areas. It is our opinion that this is a conservative approach since the 1978 landslide debris has moved to a much lower slope position onto a failure plane that is shallower than the failure plane at the headscarp, and thus, should have less driving force now than the landslide had before the 1978 movement. Additionally, the western upper portion of the landslide has been stabilized with a keyed and benched fill and our recommendations for development of the upper portion of 1925 ECDLL include retaining an upper portion of the 1978 landslide, improved surface drainage that is intended to capture most of the rainfall and runoff in the developed area and the installation of horizontal drains to intercept and capture subsurface drainage near the upper portion of the 1978 landslide. Thus, it is our opinion that the 1978 landslide debris, while likely to experience slow creep in the future, is less likely to experience large-scale, deep-seated landslide movement similar to that which occurred in 1978. Even if it were to move more rapidly, it should not adversely impact the proposed development at 1925 ECDLL, since we have modeled the site assuming that this material has already completely evacuated.

5.1.3 Seismic Ground Shaking – The proposed residential development will likely experience strong and possibly violent seismic ground shaking during its 75-year design life, which has the potential to adversely impact residential structural integrity as well as to initiate landslides. Provided that the structure is designed to current building codes and the foundation design recommendations of this report are incorporated into the design, construction and operation/maintenance of the residence, seismic ground shaking presents a low risk to the proposed residential development.

5.1.4 Expansive Earth Materials – Potentially expansive earth materials, both surficial soil and bedrock materials, could produce adverse impacts to residential structures not designed to resist forces generated by expansion. Expansive surficial soil materials can experience volume increases with an increase in moisture, and volume decreases as moisture conditions decrease, resulting in the potential for distress to structures built on shallow foundations. Expansive surficial soil materials typically are susceptible to rapid soil creep, that can transport shallowly founded structures downslope over time. In addition, expansive bedrock materials can exert uplift pressures on shallowly founded piers. Provided the foundation recommendations of this report are incorporated into the design, construction, and operation/maintenance of the residential structure and appurtenances, expansive earth materials present a low risk to the development.

5.1.5 Tsunami/Wave Run-up Hazards – The proposed development envelope is between elevations 90 feet and 130 feet, and thus, outside of the anticipated tsunami/wave run-up inundation zones during the 75-year economic life of the residence. A more thorough discussion of the tsunami and wave run-up hazards is presented in the Geosoils, Inc. report (2012) for this parcel.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 Foundation Design Considerations

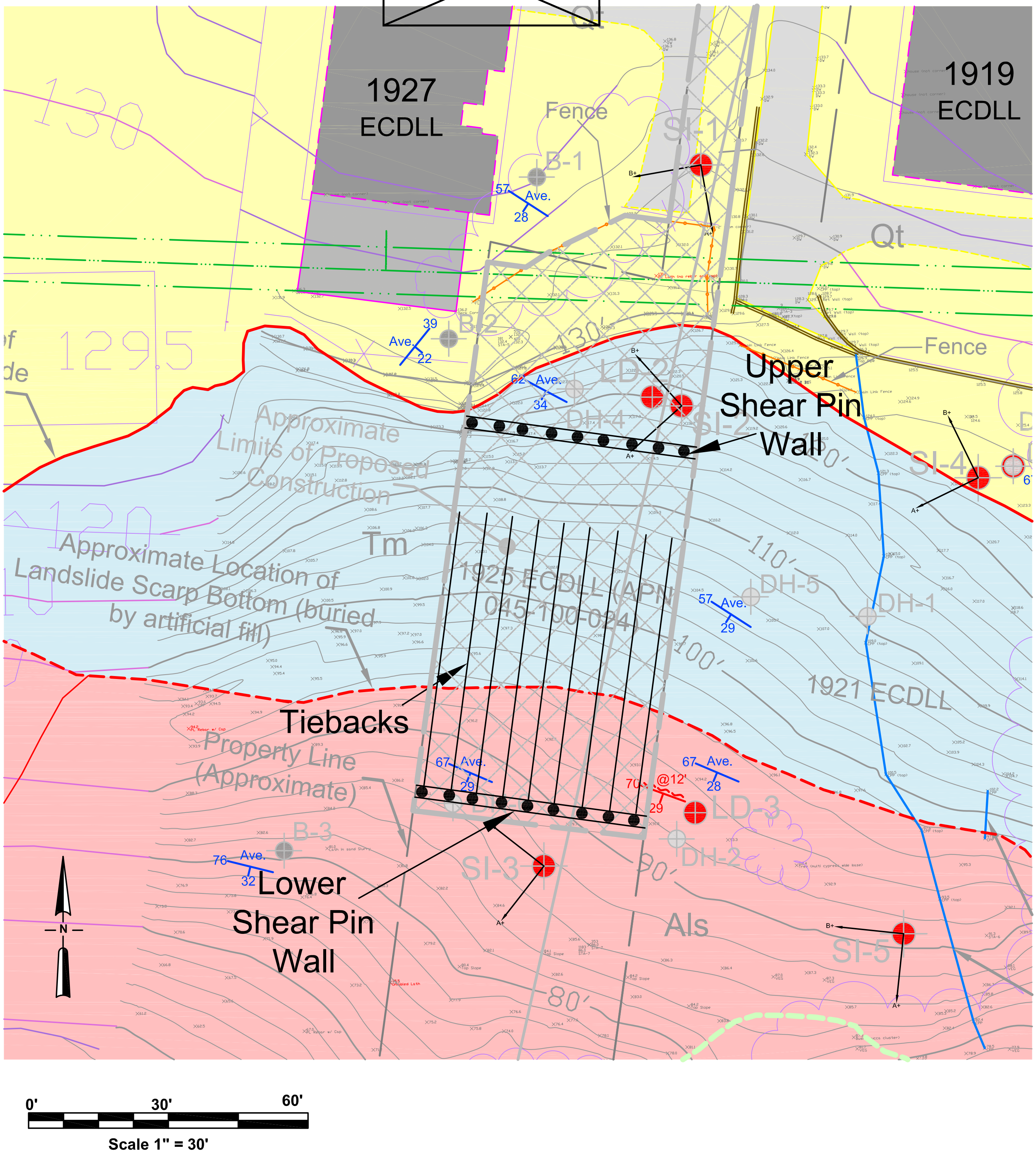
The principal factors affecting foundation selection are the variable thickness of landslide debris underlying the downslope side of the residential reuse area on the subject parcel, the weaker weathered bedrock, and the potentially weaker bedding planes of both the unweathered and weathered bedrock. We have provided recommendations for protecting the proposed residential reuse envelope (which is primarily upslope of the 1978 landslide) with two (upper and lower) rows of shear pins designed to minimize potential landslide impacts. The lower row of shear pins will include one row of tiebacks, whereas the upper row will not need to be equipped with tiebacks. The upper shear pin row is shown to be at approximate elevation 113 feet, but can be moved upward or downward slightly to accommodate the residence foundation layout (see Figures 7 and 8, Conceptual Slope Stabilization Plan and Conceptual Slope Stabilization Cross Section A-A', respectively). In addition, upslope of the tied-back row of shear pins, we are recommending a drilled, cast-in-place pier and grade beam foundation system for the proposed residence with piers extending a sufficient depth (20 feet) into intact bedrock.

6.2 Foundation Design Criteria

6.2.1 Cast-in-Place Drilled Piers - The residence and garage should be supported on reinforced concrete piers. The drilled, cast-in-place piers should derive vertical support from adhesion (skin friction) in competent, intact bedrock as determined in the field by the Project Engineering Geologist/Geotechnical Engineer at the time of construction. Residential design may utilize the upper shear pins as part of the foundation support, as deemed appropriate by the Project Structural and Geotechnical Engineers. Piers should be sized according to the following criteria:

Vertical Capacity - minimum three (3) pier-diameter spacing apart
Minimum pier diameter________________________________18 inches
Minimum pier penetration into competent weathered bedrock____20 feet

Allowable adhesion (skin friction), for reinforced concrete dead plus live loads:
In weathered bedrock__________________________________475 psf

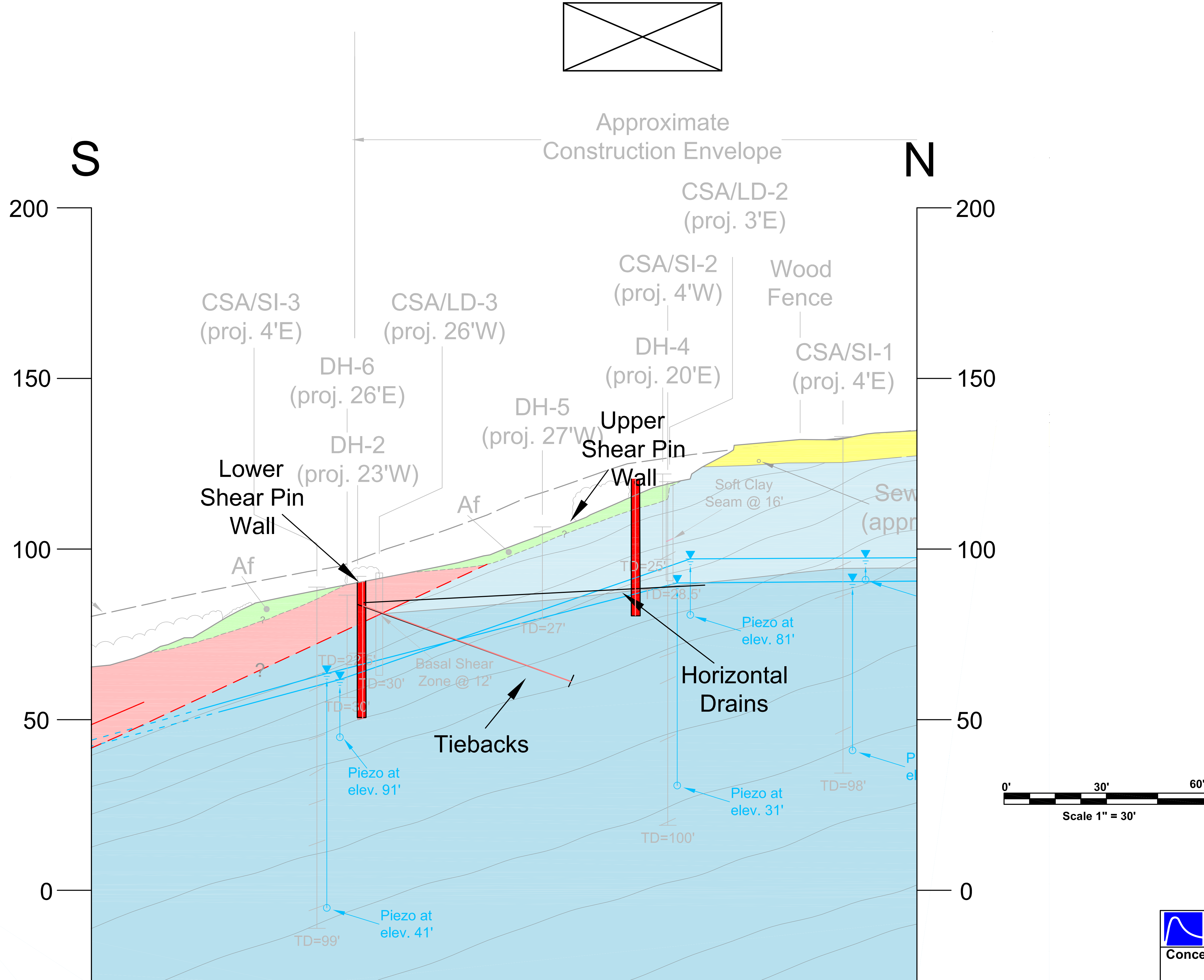


COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

CONCEPTUAL SLOPE STABILIZATION PLAN

1925 El Camino De La Luz
Santa Barbara, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=30'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	7



Cotton, Shires and Associates, Inc.
Consulting Engineers and Geologists

Conceptual Slope Stabilization Cross Section A-A'

1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
JD	1"= 30'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	8

<u>Lateral Passive Resistance</u> - piers [equivalent fluid pressure applied over an effective width of two (2) pier diameters]
Below 2 feet in weathered bedrock material____________________450 pcf

The above adhesion value (skin friction) can be increased by 1/3 for seismic loading and should be decreased by 1/2 for uplift. The upper portion of the piers should be formed to create vertical surfaces, and "mushrooming" of pier tops and over-pours around grade beams should be prevented. Drilled pier holes should be machine cleaned of all loose material prior to the placement of steel and concrete. Piers should be steel reinforced with a cage including a minimum of 4, No. 5 bars vertical (with greater reinforcement as required by the Project Structural Engineer). Casing could be necessary to prevent caving, especially in soils or landslide debris.

Water may be present in the pier holes, consequently, prior to placing concrete, the water should be pumped out until the pier holes are dry, or the concrete should be poured by tremie methods to displace the water. All pumped water and/or concrete overspill should be collected so as not to run freely across the ground surface and be disposed of offsite and outside of the coastal zone. All piers should be connected at their tops by a continuous structural slab/mat that in turn will support the structure.

6.2.2 <u>Shear Pins</u> – Shear pins should have a minimum diameter of at least 30 inches, and be at least 40 feet long (deep). In addition, the shear pins on the lower row should extend a minimum of 30 feet into unweathered bedrock or beneath the pad subgrade (whichever is deeper). Both shear pin rows (upper and lower) should consist of drilled, cast-in-place reinforced concrete piers that derive passive resistance to lateral forces in firm bedrock material, and be spaced at maximum 6 feet on-centers. Our analysis indicates that the shear pins should be designed to provide a minimum landslide resistant shear capacity of 40 kips (6.67 kips/ft) applied as a point load at a depth of 15 feet below top of shear pin for the lower row and a minimum landslide resistant shear capacity of 50 kips (8.33 kips/ft) 20 feet below top of shear pin for the upper row, or as a uniform load of 444 psf applied over a depth of 15 feet for the lower row and 417 psf applied over a depth of 20 feet for the upper row (analyze for both types of loading separately, and use the most critical case for design for each row).

The lateral loads can be resisted by passive pressure against the side of the shear pins using the Lateral Passive Resistance recommendation provided in Section 6.2.1, Cast-in-Place Drilled Piers, in the preceding recommendations, and tiebacks as described in the following recommendations. A traffic surcharge of 250 psf uniform pressure should be included and applied against the top 10 feet of the piers and shear pins where the driveway/garage is within a 1:1 projected line up from a depth of 10 feet. Shear pins can be constructed using either wide flange steel beams or reinforcing bars (minimum of 9, No. 9 evenly spaced vertical bars encased by No. 3 spiral with a 3-inch pitch or greater reinforcement as required by the Project Structural Engineer).

6.2.3 **Tiebacks** – Our analysis indicates that the lower shear pin row should be equipped with at least one row of tiebacks located 7 feet below the existing ground surface and have a design capacity of 100 kips, and be tested to 1.33 times the design load. The tiebacks should be declined 20 degrees upslope and into the hillside, have an unbonded length of roughly 35 feet and have a minimum bonded length of 30 feet (or greater as determined by the tieback contractor in order to achieve design and testing capacities) in the unweathered bedrock, and should not extend beyond the east-west property line of 1925 El Camino De La Luz with 1927 El Camino De La Luz which is 117 feet landward of the lower shear pins. The tiebacks should be structurally connected to the shear pins and be double corrosion protected. The design adhesion in the bonded zone should be determined by the tieback contractor.

6.3 Mat Floor Foundation

For a mat foundation, the subgrade should be prepared as recommended under Site Grading (Section 6.4). The mat should be at least 12 inches thick and reinforced with minimum No. 4 steel reinforcing bars at maximum 16 inches on center, both ways, and crack control joints should be provided at maximum 12-foot intervals, both ways. Steel reinforcement may be increased and expansion joints may be added as required by the Project Civil or Structural Engineer.

6.4 Site Grading

Based on our field investigation, shallow grading excavations should be within the capabilities of heavy-duty excavation equipment (i.e., excavators, dozers, and large drill rigs); however, deeper excavations may require "ripping" and/or a "hoe-ram" to excavate. It should be noted that we encountered high blow counts in our small-diameter borings and very difficult drilling conditions in the large-diameter borehole exploration in the unweathered bedrock material.

6.4.1 Site Preparation - All loose material, vegetation, concrete, large rocks, debris, and other deleterious material, without limit, should be stripped and removed from the development envelope on the parcel, for disposal offsite and outside the coastal zone pursuant to applicable entitlement or license. In areas on the parcel to be filled, the exposed surface should be scarified to at least an 8-inch depth, moisture conditioned to at least optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM D-1557-12. The subgrade beneath all fills should be keyed and benched as the fill is placed and brought upslope.

6.4.2 Compacted Fill – Excavated on-site material can be re-used as compacted fill provided it is free of organic matter and material (rocks) larger than 4 inches in diameter. Imported fill should be free of organic material and be certified weed free; it should contain no material larger than 4 inches and should have a plasticity index (P.I.) of less than 16. The fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to at least optimum moisture content, and compacted to at least 95 percent relative compaction beneath structures, slabs and within 18 inches of the aggregate baserock for pavements, and 90 percent relative compaction elsewhere based on ASTM D-1557-12.

6.4.3 Utility Trench Backfill - Utility trenches should be backfilled with approved, on-site soil. Bedding materials for pipes should be graded and placed in accordance with the manufacturer's recommendations. The backfill should be compacted to at least 90 percent relative compaction based on based on ASTM D-1557-12. Equipment and methods should be used that are suitable for work in confined areas without damaging trench walls or conduits.

6.4.4 Cut Slope Design – During the dry season, temporary cut slopes of 1.5:1 (H:V) in soils and 1:1(H:V) in bedrock should be satisfactory provided that they are inspected and approved by our field representative at the time of construction and monitored daily during construction. However, due to the dip slope bedding planes, some cuts may not be stable, and may require shoring regardless of inclination. Excavation methods, shoring, bracing and safety of excavations are the responsibility of the contractor. All excavations should comply with applicable local, State and Federal safety regulations.

6.5 Retaining Wall Designs

The following section provides our recommendations for design of site retaining walls.

6.5.1 <u>Retaining Walls</u> – Retaining walls should be supported on drilled, cast-in-place piers and designed according to the Foundation Design Criteria (Section 6.2.1) provided above. The retaining walls that are free to rotate should be designed to resist an active lateral equivalent fluid pressure of 50 pounds per cubic foot (pcf) for the existing slope inclination (we should be contacted in the event that backfill inclinations will exceed the existing 2.25:1 slope). The above active lateral fluid pressures should be increased by 50% for walls that are restrained from rotation (residential walls). The lateral loads on the retaining wall can be resisted by passive pressure against the sides of the piers using the lateral passive resistance provided both in foundation design criteria, above. For seismic loading, a dynamic resultant force acting at 1/3H up from the bottom of the wall and equal to an equivalent fluid pressure of 16 pcf should be applied to all residential retaining walls greater than 5 feet in height and any site walls located within a horizontal distance to the residence of the wall height or less.

6.5.2 <u>Backdrain</u> - Backdrains should be constructed behind all retaining walls. The backdrain should consist of a minimum 12-inch wide continuous blanket of either Caltrans Class 2 Permeable Material or 3/4-inch x 1/2-inch clean crushed drainrock enclosed in Mirafi 140N (or approved equivalent) filter fabric, and extended to within 1 to 1-1/2 feet of the ground surface where an impervious fill and/or asphaltic concrete cap should be placed. A minimum 4-inch diameter PVC Schedule 40 perforated drain pipe should be placed near the bottom of the drainrock (perforations down), surrounded by a minimum of 4 inches of drainrock with at least 2 inches of drainrock underlying the pipe. All backdrain pipes should be sloped to drain at a minimum of 1/2 percent and be collected in 4-inch diameter, non-perforated Schedule 40 PVC pipes which are sloped a minimum of 2 percent and discharged away from the landslide and in a suitable area which won't result in erosion.

6.6 Slabs-on-Grade and Concrete Flatwork

Slabs-on-grade and concrete flatwork subgrades should be prepared as recommended in Site Grading, above. Slab-on-grade floors, including the garage, should be directly underlain by at least 6 inches of clean, crushed drain rock (100 percent passing the 3/4-inch sieve; 0-2 percent passing the No. 4 sieve, and 0 percent passing the No. 200 sieve) except in areas of the bottom floor subdrain which should have a thicker section (See Drainage section below for mat subdrain design). For damp-proofing of the slab, a layer of Moistop Underslab Vapor Retarder or Stegowrap should be provided over the capillary break (gravel or crushed rock).

Concrete flatwork (sidewalks, patios, etc.) should be supported on at least 6 inches of moist, compacted Caltrans Class 2 Aggregate Base rock material. The 6 inches of compacted base rock material should, in turn, be underlain by compacted fill or firm natural material.

Slabs and flatwork should be steel reinforced with at least No. 4 bars at 18 inches on centers each way (or greater reinforcement as determined by the Project Structural Engineer), and provided with crack control joints at maximum 10 feet on centers, both ways.

6.7 Drainage

Because of the detrimental influence of water as it interacts with soil, bedrock, foundations, pavements, and cut and fill slopes, it is important that surface water be controlled. Grades should be sloped to drain at a minimum of 2 percent for a distance of at least 10 feet out from structures with runoff directed into an appropriate catch basin/storm drain system. All roof runoff should be collected in gutters with downspouts tied into tightline pipes (Schedule 40 PVC) that also discharge into a catch basin/storm drain. The catch basin/storm drain should discharge into the property and City storm drainage system.

Where concrete curbs are used to isolate landscaping in or adjacent to pavement areas, we recommend that the curb extend a minimum of 8 inches into low permeable material below the baserock to provide a barrier against the migration of landscape water into the pavement section.

6.7.1 Sub-Floor Mat/Slab Subdrains – The mat/slab-on-grade floor should be underlain by a minimum 6-inch thick blanket of clean, free-draining crushed rock or gravel as specified in Slab-on-Grade and Concrete Flatwork sections, above. The blanket subgrade should be cut to drain (hydraulically connected) to one of the sub-floor subdrains which should be spaced at minimum 30-foot intervals and extend across the entire slab. The sub-floor slab subdrains should consist of a 4-inch diameter perforated Schedule 40 PVC pipe sloped a minimum of 1/2 percent and placed in a minimum 12-

inch wide, 6- to 18-inch deep or deeper (depending on the dimensions of the sub-floor) trench filled with crushed rock or gravel and a sheet of filter fabric separating the gravel from the blanket subgrade. There should be 2 inches of drainrock in the bottom of the trench, below the pipe. The subdrain pipes should be collected in 4-inch diameter, non-perforated Schedule 40 PVC pipes sloped a minimum of 2 percent and discharged either directly into the storm drain system by gravity outlet, or drained into a sump(s) equipped with a pump(s) which in turn flow into the property storm drainage system. The retaining wall backdrains should also be collected and drained in a similar manner as the sub-floor slab subdrain, or combined, if preferred. Clean-outs should be provided at both ends of each the sub-floor slab subdrain. Surface water should not be discharged into subdrain pipes.

6.8 Seismic Design

A peak ground acceleration of 0.44 g should be anticipated for design purposes.

Based on our geotechnical investigation, the site location and our interpretation of the 2007 CBC documents related to Earthquake Loads (CBC Section 1613), we are providing the following parameter recommendations from the corresponding figures and tables:

Parameter	Referenced Table/Figure/Eqn.	Value
Site Classification	1613.5.2	C
Mapped Spectral Acc. 0.2 Sec. (g)	1613.5(3)	$S_S = 1.890$
Mapped Spectral Acc. 1 Sec. (g)	1613.5(4)	$S_1 = 0.711$
Fa – Site Coefficient	1613.5.3(1)	1.0
Fv – Site Coefficient	1613.5.3(2)	1.3
Seismic Design Category	1613.5.6	D
$S_{MS} = F_a S_s$	16-37	1.890
$S_{M1} = F_v S_1$	16-38	0.924
$S_{DS} = 2/3\ S_{MS}$	16-39	1.260
$S_{D1} = 2/3\ S_{M1}$	16-40	0.616

6.9 Horizontal Drains

We recommend horizontal drains be installed along the lower shear pin wall. The drains should be spaced approximately 18 feet apart to avoid the upper shear pin wall, be inclined 2-degrees upward upslope, and extend a minimum of 100 feet into the slope. The drain outlets should be connected to tightline collector pipes and discharge into the newly established storm drain system designed to capture the residential runoff. The horizontal drains should be equipped with cleanout access ports, and the drains should be periodically flushed and inspected at a maximum of 5-year intervals.

7.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or merchantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

Any recommendations and/or design criteria presented in this report are contingent upon our firm being retained to review the final drawings and specifications, to be consulted when any questions arise with regard to the recommendations contained herein, and to provide testing and inspection services for earthwork and construction operations. Unanticipated soil and geologic conditions are commonly encountered during construction and cannot be fully determined from existing exposures or by limited subsurface investigation. Such conditions may require additional expenditures during construction to obtain a properly constructed project. Some contingency fund is recommended to accommodate these possible extra costs.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are called to the attention of the project engineer and incorporated into the plans. Furthermore, it is also the responsibility of the owner, or of his representative, to ensure that the contractor and subcontractors carry out such recommendations in the field.

8.0 REFERENCES

8.1 Documents/Maps

Aero Service Corporation, 1965, Topographic Base Map, City of Santa Barbara and the County of Santa Barbara Flood Control and Water Conservation District, 1"=200'.

Blake, T. F., 1989, EQFAULT: A computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults, IBM-PC Version, Users Manual, July, 1989.

Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., June 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California.

Buena Engineers, 1983, Landslide Treatment at 1933 El Camino De La Luz, Santa Barbara, California, May 5, 1983.

Buena Engineers, Inc., 1983, Soils and Geologic Report, Doolittle, May 5, 1983.

California Coastal Commission, 1984, CDP 4-84-17 (Doolittle et al.)

California Department of Conservation Division of Mines and Geology, 1997, Special Publication 117: Guidelines for Evaluating and Mitigating Seismic Hazards in California.

California Department of Conservation Division of Mines and Geology, 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California.

California Division of Mines and Geology, 1996, California Fault Parameters, DMG Open File Report 96-08.

Campbell-Geo, Inc., 2007, Geologic/Preliminary Geotechnical Evaluation, Monteabaro Property, 1927 El Camino De La Luz, March 5, 2007.

City of Santa Barbara, 2006, Draft EIR on the 1837 1/2 El Camino de la Luz Residence.

City of Santa Barbara, 1978, Geologic, Seismic Hazard, Liquefaction Hazard, Soil Creep and Expansive Soil Hazards, and Erosion Hazard maps, prepared by Michael Hoover, October 17, 1978, Scale = 1:12,000.

City of Santa Barbara, 1979, Seismic Safety Element of the General Plan (1979).

City of Santa Barbara, 2004, Local Coastal Plan, 1981, Amended November 2004.

City of Santa Barbara, 2007, Neighbor's Photo Album of 1978 ECDLL Landslide, submitted in 2007 to City Community Development Department.

City of Santa Barbara, 2011. General Plan Update.

Cook, L. P., 2006, Boundary and Topographic Survey, 1925 El Camino De La Luz.

Dibblee, T.W., 1986, Geologic Map of the Santa Barbara Quadrangle, Santa Barbara County, California, Dibblee Foundation Map DF-06, Scale = 1:24,000.

Fairchild Aerial Photograph Collection, Whittier College, 1929, 1938, and 1950.

Geosoils, Inc., 2012, Wave Runup & Coastal Hazard Analysis, 1925 El Camino De La Luz, Santa Barbara, California (APN 045-100-024).

Gurrola, L.D., et al, 2003, Active Tectonics and Earthquake Hazards of the Santa Barbara Fold Belt, Southern California Earthquake Center Field Trip Guide Book, September, 7.

Hoek, E., and E. T. Brown, 1997, Practical Estimates of Rock Mass Strength: International Journal of Rock Mechanics and Mineral Science, vol. 34, no. 8, p. 1165-1186.

Hoek, E. and Bray, J.W., 2001, Rock Slope Engineering, 3rd Edition, The Institute of Mining and Metallurgy.

Hoover, Michael, 2007, Letter to City of Santa Barbara on behalf of his clients November 2.

Idriss, I. M., 1987, Lecture Notes, Presentation at the EERI Course on Strong Ground Motion, April 10-11, 1987, Pasadena, California, <u>in</u> Singh, J. P., Course Organizer, Strong Ground Motion, Seismic Analysis, Design, and Code Issues: Earthquake Engineering Research Institute, 14 pp.

Johnson, M. J., 2002, Establishing Development Setbacks From Coastal Bluffs, in Magoon, Orville et al; Proceedings, California and the World Ocean 2002, Reston, Virginia: American Society of Civil Engineers.

Joyner, W. B., and Boore, D. M., 1981, Peak horizontal acceleration and velocity from strong-motion records including records from the 1979 Imperial Valley, California, earthquake: Bulletin of the Seismological Society of America, v. 71, p. 2011 - 2038.

Keller, E.A., Gurrola, L.D., 2000, Earthquake Hazard of the Santa Barbara Fold Belt, California, Institute for Crustal Studies, U.C. Santa Barbara, California; NEHRP Award 99HQGR0081, and SCEC Award 572726.

Krinitzsky E.L., F.K. Chang and O.W. Nuttli, 1988, Magnitude-Related Earthquake Ground Motions: Bulletin of the Assoc. of Engineering Geologists, Vol. XXV, No. 4, pp. 399-423.

Minor, S.A., et al., 2006, Preliminary Geologic Map of the Santa Barbara Coastal Plain Area, Santa Barbara County, California, U.S. Geological Survey Open File Report OFR 02-136.

Pacific Materials Laboratory, Inc., 1978, Preliminary Landslide Investigation, Sea Cliff Property El Camino Del La Luz, March 6, 1978.

Padre Associates, Inc., 2006, Geotechnical Study, 1921 El Camino De La Luz.

Padre Associates, Inc., 2007, Evaluation of Bluff Setback, 1921 El Camino De La Luz, Santa Barbara, California; May 9, 2007.

Padre Associates, Inc., 2008, Retraction of Geotechnical Study, 1921 El Camino De La Luz, Santa Barbara, California; April 17, 2006.

Peak Surveys, Inc., Slope Repair (Grading) Plan, Doolittle Residence, 1933 El Camino de la Luz, 1984.

Rocscience, 2007, RocLab version 1.031.

Scepan, Joseph, 2012, Coastal Bluff Position Aerial Photographic Analysis, 1950-2010, 1925 El Camino de la Luz, Santa Barbara, California.

Smith, R.J., 1980, Engineering Geology of Subsurface Soils, 1927 Camino De La Luz, July 7.

Terzaghi, K., and Peck, R.B., 1967, Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc.

Towill, Inc., and Penfield and Smith Engineers and Surveyors, 1997; Topographic Map of the City of Santa Barbara, Public Works Department.

U.S. Coast Survey, Survey of Point Near Santa Barbara for a Light House Site November, 1854, Register 470, 1:10,000 scale.

U.S. Coast and Geodetic Survey, Santa Barbara and Approaches, California, 1887 (October, 1898), Sheet 5261, 1:20,000 scale.

WRA, Inc., 2012. Biological Reconnaissance Report, 1925 El Camino de la Luz, Santa Barbara, California.

Wyllie, D.C., and Mah, C.W. 2004, Rock Slope Engineering, Civil and Mining, 4th Edition, Based Upon 3rd Edition by E. Hoek and J. Bray.

U. S. Department of the Navy, 1982, Design Manual Soil Mechanics, Foundations, and Earth Structures, NAVFAC DM-7.2.

U.S.G.S. Seismic Hazard Curves and Uniform Hazard Response Spectra Program, Version 5.0.9a, dated 02/10/2011.

U.S. Soil Conservation Service, Soil Survey of Santa Barbara County, California, South Coastal Part, February 1981.

Weaver, Donald W., February, 1978 El Camino de la Luz landslide (vertical aerial, Fig. 17.9).

Weaver, Donald W., 1982, Preliminary Landslide Hazards Evaluation, prepared for Mr. Robert Doolittle, 1933 El Camino de la Luz, Santa Barbara, California 93109, May 6.

8.2 Vertical Aerial Photographs (Stereo-pairs)

Date	Source	Flight No.	Scale
1929	Fairchild Collection	C-430-A-13, 14	1″=2,000′
1938	Fairchild Collection	C-4950	1″=2,000′
1950	Fairchild Collection	C-14500	1″=600′
1962	U.C. Santa Barbara	HA-01-80, 81	1″=1,000′
1975	U.C. Santa Barbara	HB-XQ-240, 241	1″=1,000′
1978	USDA	06083-178-7, 8	1″=1,000′

1983	Pacific Western	PW-SB-5-2, 3	1″=2,000′
1995	Pacific Western	PW-55010-25, 26	1″=1,000′
2005	Pacific Western		1″=1,000′

8.3 Oblique Aerial Photographs (Single-frame)

Date	Source
1972	California Department of Navigation and Ocean Development
1972	Coastal Records Project, California Coastline.org
1978	Weaver (1978)
1978	Coastal Records Project, California Coastline.org
1979	Coastal Records Project, California Coastline.org
1987	Coastal Records Project, California Coastline.org

APPENDIX A

FIELD INVESTIGATION

Topographic Surveying and Geologic Mapping
Logs of Small-Diameter Exploratory Borings
Logs of Large-Diameter Exploratory Borings

APPENDIX A

FIELD INVESTIGATION

Surveying and Geologic Mapping

The surface conditions of 1925 El Camino De La Luz were investigated by our engineering geologists and geotechnical engineers from 2008 to the present using total station theodolite surveying equipment, measuring tape and pace, Brunton compass and hand-level techniques. Cotton, Shires and Associates, Inc. (CSA) generated an original, detailed topographic base map of the parcel, with 2-foot contour intervals, in 2010 with total station theodolite surveying techniques. Outside of the parcel, we augmented our topography with the City-County topographic base map of 1997. This 2010 base map was used for our topographic profiling, engineering geologic mapping, and slope stability analysis.

Small-Diameter Exploratory Boreholes

The subsurface geologic conditions at 1925 El Camino De La Luz were explored by excavating three small-diameter boreholes in May 2011. In addition, two additional small-diameter boreholes were located on the adjacent parcel at 1921 El Camino De La Luz during the same time period. The small-diameter boreholes were excavated by Britton Drilling of Campbell, California, who primarily utilized mud-rotary, continuous coring methods; however, the upper portions of the boreholes within artificial fill, weathered bedrock and landslide debris were drilled using hollow-stem auger techniques. Retrieved cores from the borings were brought back to our laboratory and logged in detail to augment our on-site logging. The cores remain in our laboratory for future inspections, as necessary. Detailed logs of these borings are presented in this appendix (following the text). These logs depict our interpretation of the subsurface conditions at the dates and locations indicated. It is not warranted that they are representative of subsurface conditions at other times and locations. The contacts on the logs represent the approximate boundaries between earth materials, and the transition may be gradual. Representative samples of earth materials were collected for subsequent laboratory identification and testing (see Appendix B for a summary of laboratory testing).

Large-Diameter Exploratory Boreholes

Two large-diameter boreholes (24-inch diameter) were excavated by RC Drilling of Thousand Oaks, California in October, 2011 for the purpose of investigating the subsurface conditions of 1925 El Camino De La Luz. A third large diameter borehole was excavated on the adjacent property at 1921 El Camino De La Luz and was used to aid in correlation of bedrock units at 1925 El Camino De La Luz. These boreholes were downhole logged by our staff and principal engineering geologist in October, 2011. We selectively sampled critical earth materials, such as the weakest claystone interbeds in the upper borehole and the sheared materials in the lower borehole, for laboratory testing. The boreholes were backfilled with the native spoils and compacted using tamping methods with the drill rig Kelly bar. Geologic logs of the boreholes are presented in this appendix.

APPENDIX B

LABORATORY TESTING

Table B-1, Summary of Laboratory Test Results
Figure B-1, Summary of Atterberg Limits
Figure B-2, Undrained, Consolidated Triaxial Compression Strength Tests
Figure B-3, Summary of Torsional Ring Shear Tests.

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Felkay, 1925 ECDLL **Boring** B-1/SI-1

Location LANDSCAPED MEDIAN BETWEEN DRIVEWAYS **Project No.** G0058

Drilling Contractor/Rig Britton Exploration/ CME 550 TRACKED RIG **Date of Drilling** 5/10/11

Ground Surface Elev. ~132' **Logged By** JD **Hole Diameter** 6"ø AUGERS, 3.5"ø HQCORE, 4 7/8"ø TRICONE

Surface BARE SOIL/VEGATATION, A+DIRECTION: SIDE **Weather** CLEAR, BREEZY, COOL

TERRACE DEPOSITS?/ ARTIFICIAL FILL (0–5.5')

TERRACE DEPOSITS (5.5–7.0')

WEATHERED MONTEREY FORMATION (7.0' and below)

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
			0.0'-5.5': SILTY CLAY w/ SAND; DAMP TO MOIST, FEW WHITE DENDRITIC MINERALIZATIONS, FIRM TO STIFF, MODERATE TO HIGHLY PLASTIC, ABUNDANT ROOTS.					HA		DRILLER: PAUL HELPER: SERGIO START DRILLING @ 8:30AM
2		CL	0.0'-5.5': ARTIFICIAL FILL/ TERRACE DEPOSITS?	2 1			3 9 12	MC	15"/18"	w/ HOLLOW STEM AUGERS.
4								HA		
			5.5'-7.0': TERRACE DEPOSITS							
6		SC	5.5-7.0': FINE TO MEDIUM SAND w/ CLAY; MOIST, MEDIUM RED BROWN, FEW 1/2" PEBBLES (ROUND), FRIABLE, WEAK.	4 3			9 13 10	MC	15"/18"	
			7.0'-98.0': MONTEREY FORMATION.					HA		
8			7.0'-38.0': WEATHERED MONTEREY FORMATION; FINE LAMINATED SHALE, DAMP TO MOIST, VARIGATED BUFF TAN TO BLACK, BRITTLE, HARD SHALE LAMINATIONS (PORCELAIN LIKE) ALTERNATING w/ BROWN TO BLACK WEAKER SHALE LAMINATIONS, MODERATE TO DEEPLY WEATHERED, WEAK TO FRIABLE, LOW TO MODERATE HARDNESS, INTENSELY	6 5			3 9 50/4"	MC	15"/18"	
10			10.0'-11.5': FEW OFFSET LAMINATIONS, DEEPLY WEATHERED TO CLAY, VERY PLASTIC.	8 7			6 14 13	MC	14"/18"	
12										
14		SHALE						HA		
16			15.0'-16.5': HARDER SHALE, WEAK, BRITTLE, DAMP, FINELY LAMINATED	10 9			13 23 50/5"	MC	13"/18"	
18								HA		
20			20.0'-21.5': FINELY LAMINATED, HIGHLY PLASTIC, VERY CLAYEY VARYING TO BRITTLE, DAMP.	12 11			24 14 17	MC	13"/18"	9:35AM
22								HA		-SET 4.5"ø CASING TO 25'
24										
26			25.0'-26.5': DEEPLY WEATHERED SILTY CLAYSTONE, HIGHLY OXIDIZED, HIGHLY PLASTIC, SOIL-LIKE, MOIST, VERY STIFF, VARIGATED LIGHT GRAY AND REDDISH ORANGE.	14 13			7 12 29	MC	17"/18	10:15AM: SWITCHING TO MUD ROTARY DRILLING 11:30AM-1:15PM: DRILL RIG
			26.5'-28.0': DARK GRAY BROWN w/ YELLOW ORANGE OXIDIZED STREAKS, SILTY CLAYSTONE, LAMINATED, MOIST, HIGHLY PLASTIC.	1				HQ	0.5'/1.5'	SHUT DOWN; NEEDS A NEW FLUID PRESSURE GAUGE.
28				2				HQ	3.5'/5.0'	2:00PM: START CORING

Project Felkay/G0058 **Date** 5/10/11 **Boring** B-1/SI-1

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
28.0–33.0 (WEATHERED MONTEREY FORMATION)			28.0'-33.0': HARD, BRITTLE, LAMINATED SHALE, DOLOMITIZED, VERY FRACTURED	2				HQ	3.5'/5.0'	2:32 PM @ 33'
33.0–38.0		25° DIP, BUFF TO TAN	33.0'-38.0': HARD, STRONG SHALE, BUFF TO TAN W/ GRAY TO BLACK LAMINATIONS, CLOSELY FRACTURED. POOR RECOVERY. VERY HARD CORING RUN W/ A FEW FASTER SPOTS. FINELY LAMINATED, FEW OXIDIZED STREAKS.	3				HQ	1.5'/5.0'	3:15PM @ 38'
38.0–43.0 (UNOXIDIZED MONTEREY FORMATION); 40: VW PIEZO		TAR SEAMS	38.0'-43.0': BOTTOM 3.5' OF CORE INTACT. DARK BROWN, CLAYSTONE W/ ABUNDANT PETROLEUM/TAR SEAMS IN FRACTURES, HARD, MODERATELY STRONG, LAMINATED, LITTLE WEATHERING, CLOSELY FRACTURED.	4				HQ	4.0'/5.0' 4mins	3:40PM; @ 41': VIBRATING WIRE PIEZOMETER SENSOR S/N 1107485; 3:44PM
43.0–48.0		20° DIP	43.0'-48.0': CORE IS COMPLETELY INTACT; A FEW MECHANICAL BREAK AT THE TOP. DARK BROWN CLAYSTONE, HARD, MODERATELY STRONG, LAMINATED, FRESH, CLOSE TO MODERATELY FRACTURED, NUMEROUS TAR SEAMS ALONG FRACTURES, SOME OFFSET LAMINATIONS AT BOTTOM OF RUN.	5				HQ	5.0'/5.0' 6mins	4:10PM; 4:16PM @ 48'
48.0–53.0		TAR SEAMS	48.0'-53.0': CORE IS NEAR INTACT W/ 4 MECHANICAL BREAKS. SIMILAR TO ABOVE IN ROCK PROPERTIES. HARD, STRONG, TAN TO WHITE, LAMINATED DOLOMITIZED ZONE NEAR BOTTOM OF RUN.	6				HQ	5.0'/5.0' 10mins	4:50PM; ~~5:00PM @ 53'~~ END OF DAY; 7:30AM 5/11/11
53.0–58.0		TAR SEAMS	53.0'-58.0': CORE HAS NUMEROUS MECHANICAL BREAKS. DARK BROWN CLAYSTONE W/ BLACK DISEMINATIONS OF HYDROCARBONS (TAR SEAMS) MODERATELY HARD, MODERATELY STRONG, CLOSELY TO INTENSELY FRACTURED, LITTLE TO NO WEATHERING. TAR SEAMS FILL IN FRACTURES.	7				HQ	8:04AM; 5.0'/5.0' 12mins; 8:16AM	
58.0–63.0			58.0'-63.0': CORE IS VERY BROKEN UP. DARK BROWN CLAYSTONE, MODERATELY HARD, WEAK TO MODERATE STRENGTH, CRUSHED TO INTENSELY FRACTURED.	8				HQ	8:39AM; 4.0'/5.0' 11mins; 8:50AM	
63.0–				9				HQ	9:08AM	

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Project Felkay/G0058 Date 5/11/11 Boring B-1/SI-1

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
64–68		63.0'-68.0': CORE IS BROKEN UP, TWO PIECES ARE 8" TO 10" LONG. SILTY CLAYSTONE TO FINE SANDY CLAYSTONE W/ FEW 1/8"-1/4" PEBBLES OR CLAST, MEDIUM TO DARK BROWN, MORE VARIABLE IN ROCK PROPERTIES. SANDY SECTIONS ARE HARD, STRONG, MODERATELY FRACTURED, LAMINATED, CLAYEY SECTIONS ARE MODERATELY HARD, WEAK TO MODERATE STRENGTH, INTENSELY FRACTURED TO CRUSHED.	9 Box4				HQ	4.5'/5.0' 12mins 9:20AM	-VARIABLE RUN; SLOW AT THE START, SPEEDS UP IN THE MIDDLE, SLOW AT THE END OF RUN
68–73	CLAY SEAM	68.0'-70.0': FINE SANDY SILTY CLAYSTONE, MEDIUM BROWN, HARD, STRONG. @70.0': 0.5" THICK VERY PLASTIC, RUBBERY CLAY SEAM, YELLOW GREEN TO GREEN BROWN, HORIZONTALLY ORIENTED, MOIST, STRONG H_2S ODOR. ·BELOW: BEDDING IS DISRUPTED @ 71.5' W/ SOME PLASTIC CLAY SEAMS	10 Box 4				HQ	9:57AM 5.0'/5.0' 11mins 10:08AM	
73–78	CLAY SEAM	@ 75.0': CLAY SEAM, MUSTARD YELLOW, VERY PLASTIC, MOIST ·BELOW: MODERATELY HARD, MODERATELY STRONG, FEW FRACTURES OFFSET BEDDING LAMINATIONS, MEDIUM TO DARK BROWN.	11 Box 5			X	Ha	10:30AM 3.5'/5.0' 15mins 10:45AM	
78–81		78.0'-81.0': FRACTURED CLAYSTONE; DARK BROWN, MODERATELY HARD, WEAK TO MODERATELY STRONG, CRUSHED TO INTENSELY FRACTURED	12 Box 5				HQ	11:11AM 3.0'/3.0' 12:00PM	HARD CORING RUN; BIT NO LONGER ADVANCING @ 80' SWITCHING FROM A CARBIDE BIT TO A DIAMOND BIT
81–83	HARD	81.0'-83.0': SILTY CLAYSTONE; DARK BROWN, WELL CEMENTED, MODERATELY HARD, MODERATELY STRONG, BRITTLE, FINELY LAMINATED, UNIFORM BEDDING	13 Box 5				HQ	12:35PM 2.0'/2.0' 12:43PM	
83–88	25° DIP	83.0'-88.0': SILTY CLAYSTONE; DARK BROWN, WELL CEMENTED, MODERATE TO HARD, MODERATE TO STRONG, CLOSELY FRACTURED, FINELY LAMINATED,	14 Box6			X	HQ	1:10PM 2.5'/5.0' 20mins 1:30PM	
88–93	GRAVEL LAG; 40° DIP	88-91.0': DARK BROWN SILTY CLAYSTONE WITH BASAL GRAVEL LAG, MODERATELY HARD, WEAK STRENGTH, BRITTLE, INTENSELY FRACTURED. 91.0'-93.0': TAN TO BUFF WHITE SILTY CLAYSTONE, HARD, STRONG, WELL CEMENTED, PORCELAIN-LIKE, CLOSELY FRACTURED.	15 Box 6			X	HQ	1:50PM 28mins 4.5'/5.0' 2:19PM	@91': VIBRATING WIRE PIEZOMETER SENSOR S/N 1107564
93–98	TAR SEAMS	93.0'-98.0': MEDIUM BROWN SILTY CLAYSTONE, MODERATE HARDNESS, MODERATE STRENGTH, CLOSELY FRACTURED, FEW TAR SEAMS INFILLING FRACTURES.	16 Box 7			X	HQ	2:48PM 2.5'/5.0' 22mins 3:10PM	

VW PIEZO (at 91')

Project Felkay/G0058 Date 5/11/11 Boring B-1/SI-1

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
			TD = 98'							4:10PM: HQ RODS AND 4.5"∅ CASING OUT OF HOLE.
100			11:50AM: INSTALLED 100' OF SLOPE INDICATOR 2.75"∅ QC CASING W/ 2 GEOKON VIBRATING WIRE PIEZOMETERS TAPED TO THE SIDE OF THE CASING @ DEPTH 95' (S/N 1107564) AND 45' (S/N 1107485). 4' OF STICK UP.							4:20PM: START REAMING OUT HOLE W/ 4 7/8"∅
102										TRI CONE BIT. 4:57PM: REAMED TO 50'
104			1:00PM: PUMPED ~150 GALS OF BENTONITE/ CEMENT GROUT MIX PER: 30 GALS H_2O/ 94lb CEMENT/ 25lb BENTONITE.							7:30AM 5/12/11 START REAMING AGAIN
106										
108										
110										
112										
114										
116										
118										
120										
122										
124										
126										
128										
130										

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Felkay **Boring** B-2 / SI-2

Location 1925 ECDLL HEADSCARP AREA **Project No.** G0058

Drilling Contractor/Rig Britton Exploration/ CME 550 TRACKED RIG **Date of Drilling** 5/12/11

Ground Surface Elev. ~119' **Logged By** JD **Hole Diameter** 6"Ø HOLLOW STEM AUGER, 4 7/8"Ø TRI CONE

Surface BARE SOIL **Weather** CLEAR, WARM, BREEZY

Unit	Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
ARTIFICIAL FILL	0–2			0.0'-5.0': ARTIFICIAL FILL; DESCRIBED BELOW.					HA		2:55PM: START DRILLING DRILLER: PAUL HELPER: SERGIO
	2–3.5		AF	2.0'-3.5': ARTIFICIAL FILL COMPOSED OF MONTEREY SILTY CLAYSTONE CLASTS IN SILTY CLAY MATRIX, DAMP, OXIDIZED, DEEPLY WEATHERED MEDIUM BROWN AND YELLOW ORANGE, WEAK,				3 3 4	MC	12"/18"	-DRILLING W/ HOLLOW STEM AUGERS
	3.5–5								HA		
WEATHERED MONTEREY FORMATION	5–6.5			5.0'-100.0': MONTEREY FORMATION; AS DESCRIBED BELOW. 5.0'-6.5': COLLUVIUM/DEEPLY WEATHERED CLAYSTONE, VERY PLASTIC, NOT SHEARED	X			6 6 10	MC	16"/18"	SAMPLE #1 EXTRUDED
	6.5–8								HA		-3:15PM: GRINDING ON HARD ROCK @ 8'
	8–9.5			@ 8.0': CHERTY/DOLOMITIZED SHALE, HARD, STRONG, BUFF TO TAN.				50/3" – –	MC	3"/18	3:30PM:
	9.5–10								HA		
	10–11.5		COL/ WTHRD CLYST	10.0-11.5: DEEPLY WEATHERED CLAYSTONE, MEDIUM BROWN TO YELLOW ORANGE, SOIL-LIKE, VERY PLASTIC, MOIST, SOFT TO FIRM, BRITTLE, FRIABLE, SOME CLAYEY ZONES, SOME GYPSUM CRYSTALS	X			7 14 9	MC	10"/18'	SAMPLE #2 EXTRUDED
	11.5–15								HA		
	15–16.5			15.0'-16.5': DEEPLY WEATHERED CLAYSTONE, W/ A FEW HARDER, BRITTLE SHALE FRAGMENTS, MOIST, HIGHLY PLASTIC, SOFT TO FIRM, MUSKY SOFT, WET, NOT SHEARED BUT DEFORMED, SOME BRITTLE HARD PIECES, BUFF TO TAN, SOIL-LIKE	X			14 25 50/5"	MC	16"/16"	4:20PM SAMPLE #3 EXTRUDED
	16.5–20								HA		
	20–21.5			20.0-21.5: HARDER CLAYSTONE FRAGMENTS IN A CLAY MATRIX, MOIST TO WET, SOFT TO FIRM, HIGHLY PLASTIC DEEPLY WEATHERED CLAYSTONE, VERY CLAYEY, NOT SHEARED, WEAK, SILTY, LAMINATED.	X			2 13 7	MC	12"/18"	SAMPLE #4 EXTRUDED
	21.5–25								HA		
	25–26.5		▽	25.0-26.5: FREE WATER ON SAMPLER, WEATHERED CLAYSTONE, TAN TO BUFF AND REDDISH DARK BROWN, WET, HARDER BRITTLE FRAGMENTS AND SOFTER CLAYEY MATRIX.	6 5			12 13 23	MC	16"/18"	4:57PM
	26.5–30		-25' DTP						HA		

Project Felkay/G0058 **Date** 5/12/11 **Boring** B-2/SI-2

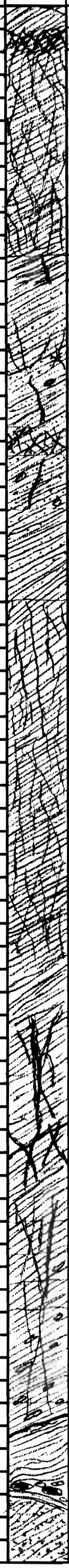
WEATHERED MONTEREY FORMATION (30'–35')

UNOXIDIZED MONTEREY FORMATION (35'–64')

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
30–35		CLAY SEAM 25° DIP; TAR SEAMS	30.0'-31.5': WEATHERED CLAYSTONE, DARK BROWN, MOIST TO WET ON FRACTURES, WEAK TO MODERATELY STRONG, LOW HARDNESS, INTENSELY FRACTURED.				50/5"	MC	5"/18"	6:30AM: SWITCH TO HQ CORING SET 4.5"Ø CASING TO 30' END OF DAY; 7:30AM 5/13/11
			* 6" CORING RUN FROM 29.5'-30.0': SILTY CLAYSTONE; MEDIUM BROWN, DAMP, MODERATE TO HARD, MODERATE STRENGTH, LITTLE WEATHERING, INTENSELY TO CLOSELY FRACTURED, FINELY LAMINATED, HYDROCARBON FILLED FRACTURES (TAR SEAMS)	RUN 2 5' BOX 1				HQ	9:20AM 3.0'/5.0' 10mins 9:20AM	8:50AM: FIRST CORING RUN TO 30' TO FINISH OUT MC RUN. CORE GOT STUCK IN THE DRILL TUBE.
35–40 (38.35: VW PIEZO)			35.0'-40.0': SILTY CLAYSTONE, DARK BROWN, LOW TO MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, DAMP TO MOIST, CRUSHED TO INTENSELY FRACTURED (FEW INTACT PIECES OF CORE), FEW 1/2"-1/4" PEBBLES AND/OR CLASTS, FINELY LAMINATED, SOME HYDROCARBON SEAMS.	RUN 3 BOX 1				HQ	10:30AM 5.0'/5.0	CORING WAS SLOW @ FIRST SPED UP IN THE MIDDLE AND SLOW @ THE END. @ 38.35': VIBRATING WIRE PIEZOMETER SENSOR S/N 1107487
40–43		BUFF TO TAN	40.0'-43.0': SILTY CLAYSTONE, DARK BROWN, MODERATE TO HARD, MODERATE TO STRONG, MODERATE FRACTURING, CONTORTED BEDDING TRANSITIONING @ 41' TO WHITE TO TAN SILICIFIED SHALE, HARD TO VERY HARD, STRONG TO VERY STRONG, LITTLE FRACTURING, TAR FILLED FRACTURES.	RUN 4 BOX 2				HQ	10:36AM 10:54AM 17mins 2.75'/3.0' 11:11AM 11:40AM 11mins 11:51AM	SLOW, HARD CORING RUN 11:11AM @ 42'. SETTING UP RECIRCULATION TUB. 11:51AM: CORED A FEW INCHES SWITCHING TO DIAMOND BIT
43–45			43.0'-45.0': SIMILAR TO ABOVE. MEDIUM BROWN LAMINATED W/ BUFF TO TAN SILTSTONE.	RUN 5 BOX 2				HQ	12:24PM 2/2' 15mins	
45–50		TAR SEAMS; 20° DIP	45.0'-50.0': SILTY CLAYSTONE; MEDIUM TO DARK BROWN, MODERATELY HARD, MODERATELY STRONG, CLOSELY FRACTURE, UNIFORMLY LAMINATED, NUMEROUS TAR SEAMS ALONG FRACTURES.	RUN 6 BOX 2				HQ	12:39PM 12:55PM 5/5' 31mins	
50–55			50.0'-55.0': SILTY CLAYSTONE; DARK BROWN, MODERATELY HARD, MODERATELY STRONG, INTENSELY TO CLOSELY FRACTURED, CONSISTENT AND UNIFORM LAMINATIONS, NUMEROUS TAR SEAMS ALONG FRACTURES	RUN 7 BOX 3				HQ	1:26PM 1:42PM 5/5' 31mins 2:06PM	
55–57		FISSILE WEAK SOFT ZONE	55.0'-57.0': WEAK, FISSILE SHALE/CLAYSTONE, DARK BROWN, CRUSHED TO INTENSELY FRACTURED.						2:23PM	SOFT ZONE @ 1' INTO RUN
57–60		25° DIP; TAR SEAMS	57.0'-60.0': STRONGER, MORE WELL CEMENTED, SILTY CLAYSTONE, DARK BROWN, MODERATELY HARD, MODERATELY STRONG, CLOSELY TO INTENSELY FRACTURED, ABUNDANT TAR SEAMS ALONG FRACTURES.	RUN 8 BOX 3				HQ	4.5'/5.0' 29mins	
60–62.5			60.0'-65.0': SIMILAR TO ABOVE.	RUN 9 BOX 4				HQ	2:52PM 3:14PM 3.0'/5.0' 29mins	
62.5–63		WEAK	62.5'-63.0': WEAK, CRUSHED, FISSILE SHALE, DARK BROWN, LOW HARDNESS							

Project Felkay/G0058 Date 5/13/11 Boring B-2/SI-2

UNOXIDIZED MONTEREY FORMATION



Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
	WEAK	64.5'-65.0': WEAK, CRUSHED SHALE, DARK BROWN, LOW HARDNESS	RUN 9 BOX 4				HQ	3:43PM	END OF DAY
								7:57AM	7:30AM 5/16/11
66, 68		65.0'-70.0': CLAYSTONE, DARK BROWN, LOW HARDNESS, WEAK STRENGTH, CRUSHED. LAST 6" ARE HARDER AND STRONGER, FINELY LAMINATED W/ A SEMI VERTICAL TAR SEAM.	RUN 10 BOX 4				HQ	4.0'/5.0' 24mins	66, 68
70	HARD							8:21AM 8:40AM	70
72, 74	25° DIP	70.0'-75.0': SILTY CLAYSTONE W/ FINE SANDY LAMINATIONS, ALTERNATING TAN TO BUFF AND DARKER BROWN LAMINATIONS, MODERATELY HARD, MODERATELY STRONG, CLOSELY FRACTURED, TAR FILLED FRACTURES (FEW), EVENLY AND UNIFORMLY LAMINATED, FEW 1/4" PEBBLES OR CLASTS.	RUN 11 BOX 4				HQ	3.0'/5.0' 25mins	72, 74
								9:05AM 9:22AM	
76		75.0'-77.0': SIMILAR TO ABOVE.							76
78	WEAK	77.0'-80.0': CLAYSTONE, DARK BROWN, CRUSHED TO INTENSELY FRACTURED, WEAK, LOW HARDNESS, LAMINATED, STRONG H_2S ODOR.	RUN 12 BOX 5				HQ	3.5'/5.0' 25mins	78
80								9:46AM 10:08AM	80
82, 84		80.0'-85.0': CLAYSTONE, DARK BROWN W/ WHITE TO TAN 1/4" LAMINATIONS, INTENSELY FRACTURED, LOW HARDNESS, WEAK STRENGTH, STRONG H_2S ODOR. LOCALLY MODERATE HARDNESS/STRENGTH.	RUN 13 BOX 5				HQ	4.0'/5.0' 32mins	82, 84
								10:41AM 10:58AM	
86, 88, VW PIEZO	FAULTED/BRECCIA; CLAY FILM	85.0'-90.0': FAULTED/BRECCIATED CLAYSTONE, DARK BROWN, INTENSELY FRACTURED, MODERATELY HARD, MODERATELY STRONG, ABUNDANT TAR SEAMS ALONG FRACTURES AND FAUTS. BEDDING LAMINATIONS ARE VERY DISRUPTED BY NUMEROUS, NEAR VERTICAL, ANASTOMOZING FAULTS, VERY BRECCIATED. @ 89': MUSTARD YELLOW TO GREEN, THIN CLAY FILM ON BEDDING PLANE, POOR RECOVERY, VERY PLASTIC.	RUN 14 BOX 5/6				HQ	4.5'/5.0' 29mins	86, 88; @ 88.35': VIBRATING WIRE PIEZOMETER SENSOR S/N 1107563
90								11:27AM 11:42AM	90
92, 94	INTACT; CLAY FILM	90.0-95.0: SILTY CLAYSTONE W/ FEW ELONGATE CLASTS, MEDIUM TO DARK BROWN, MODERATELY HARD, MODERATELY STRONG, CLOSELY TO MODERATE FRACTURING, FEW OFFSET LAMINATIONS (FAULTS), FEW TAR SEAMS ALONG FRACTURES/FAULTS. @ 94.75': MUSTARD YELLOW/GREEN CLAY FILM ALONG BEDDING PLANE, VERY PLASTIC, POOR RECOVERY	RUN 15 BOX 6				HQ	4.5'/5.0' 27mins	92, 94
								12:09PM 12:29PM	
96	SANDY; LT GRY	95.0'-100.0': TRANSITION FROM SILTY CLAYSTONE, MEDIUM TO DARK BROWN, MODERATELY HARD/STRONG, MODERATELY FRACTURED, FINELY LAMINATED TO RIP UP CLASTS/GRAVEL LAG @ 96.25' TO CLAYEY SILTY SAND, SLIGHTLY FRIABLE, LOW TO MOD. HARDNESS, WEAK TO MOD. STRENGTH, POORLY BEDDED.	RUN 16 BOX 6/7				HQ	5.0'/5.0' 15mins	96

Project Felkay/G0058 Date 5/16/11 Boring B-2/SI-2

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
		-25° DIP	@97.75' AND BELOW: FINELY LAMINATED SILTY CLAYSTONE, BUFF TO TAN, WEAK ALONG LAMINATIONS, MODERATE TO HARD, MODERATE TO STRONG, CLOSE TO MODERATE FRACTURE.	RUN 16 Box 6/7				HQ	5.0'/5.0' 15mins 12:44PM	
100			TD = 100'							100 1:00PM: START PULLING SAMPLING RODS AND 4½"Ø CASING @30'
102			5/17/11, 9:05AM: INSTALLED 100' OF 2.75"Ø SLOPE INDICATOR SLOPE INCLINOMETER QC CASING WITH TWO GEOKON VIBRATING WIRE PIEZOMETERS TAPED TO THE SIDE OF THE CASING AT 45' (S/N 1107487) AND 95' (S/N 1107563), 6.65' OF STICKUP.							102 卌 II 2:37PM: START REAMING HOLE
104										104 W/ 4 7/8"Ø TRI CONE BIT 5:50PM: REAMED TO 100'
106			10:30AM: PUMPED ~130 GALS OF BENTONITE/GROUT PER MIX: 30 GALS H_2O/94lb CEMENT/25lb BENTONITE							106
108										108
110										110
112										112
114										114
116										116
118										118
120										120
122										122
124										124
126										126
128										128
130										130

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Felkay **Boring** B-3/SI-3

Location 1925 ECDLL; DOWNSLOPE OF B-2 **Project No.** G0058

Drilling Contractor/Rig Britton Exploration/ CME 550 TRACKED RIG **Date of Drilling** 5/17/11

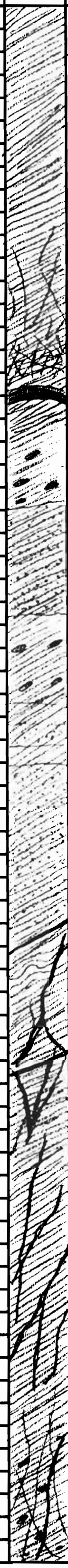
Ground Surface Elev. ~87' **Logged By** JD **Hole Diameter** 6"Ø HOLLOW STEM AUGER,

Surface BARE SOIL **Weather** CLOUDY, COOL, BREEZY

Unit	Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
ARTIFICIAL FILL	0–2		0.0'-3.0': ARTIFICIAL FILL; SILTY CLAY W/ ROCK FRAGMENTS, MEDIUM TO DARK BROWN, MOIST, PLASTIC, SILTY CLAY FILLS THE INTERSTICIES BETWEEN					HA		11:25AM: START DRILLING W/ HOLLOW STEM AUGERS
	2–3	CONC.	ROCK FRAGMENTS OF MONTEREY SHALE PIECES THAT ARE WHITE TO BUFF, WELL CEMENTED, SILICEOUS SHALE, HARD, STRONG, ONE PIECE OF CONCRETE IN DRIVE SAMPE. FILL IS LOOSE, WEAK, AND FRIABLE.				5 3 4	MC	3"/16"	11:35AM
LANDSLIDE DEBRIS	3–5							HA		
	5–6		3.0'-14.0': LANDSLIDE DEBRIS; INTENSELY FRACTURED FRAGMENTS OF MONTEREY SILTSTONE AND SHALE, DEEPLY WEATHERED, HIGHLY OXIDIZED, TAN TO BUFF AND IRON STAINED.				1 1 2	MC	10"/12"	
	7							HA		
	8–9		14.0'-99.0': MONTEREY FORMATION; AS DESCRIBED BELOW.				6 5 8	MC	12"/18"	11:50AM -12:30PM: SWITCHING TO HQ CORING SYSTEM. 8.58' LONG CORE BARREL
	9–14	CONTACT UNCERTAIN	14.0'-15.0': LIGHTLY WEATHERED CLAYSTONE; MOTTLED LIGHT GREY, YELLOW ORANGE AND DARK BROWN, MOIST, HIGHLY PLASTIC, LOW HARDNESS, WEAK, INTENSELY FRACTURED, TAR SEAMS NEAR BOTTOM OF SAMPLE ALONG FRACTURES, SOIL-LIKE, STIFF.	RUN 1 BOX 1				HQ	2:08PM 1.0'/5.0' 15mins	-LOSING FLUID CIRCULATION IN THE LOOSE FILL.
OXIDIZED MONT. FMT	14–16	VY. HARD	15.0'-15.5': SHALE; BUFF TO TAN, HARD TO VERY HARD, STRONG, WELL CEMENTED, SILICA CEMENTED, FINELY LAMINATED.	RUN 2 RUN 3				HQ HQ	2:23PM 2:40PM 3:40PM 4:45PM	-STOPPED RUN SHORT, LOSING CIRCULATION. -INSTALLED CASING TO 13' -CIRCULATION RETURNS -LOST CIRCULATION AGAIN
UNOXIDIZED MONTEREY FMT	18–20			RUN 4				HQ	0.5'/4.0' 4:49PM	-PUSHED CASING DOWN TO 15' DRILL RATE INCREASED RAPIDLY IN MIDDLE OF RUN THEN SLOWED AGAIN END OF DAY
	20–25		24.5-25.0': CLAYSTONE; DARK BROWN, BRITTLE, LOW HARDNESS, WEAK STRENGTH.	RUN 5				HQ	7:52AM 0.5'/5.0' 8:10AM	7:30AM, 5/18/11 POOR TO NO FLUID CIRCULATION, MIXING MUD TO PLUG THE LOSS.
	25–30	25° DIP	25.0'-30.0': CLAYSTONE; DARK BROWN, MODERATE HARDNESS, MODERATE STRENGTH, FINELY LAMINATED, WELL CEMENTED.	RUN 6				HQ	7:28AM 0.25'/5.0' 12mins 9:40AM	A PIECE OF CORE IS STUCK IN THE END OF THE SAMPLER SIDEWAYS BLOCKING THE SAMPLER AND PREVENTING ANY SAMPLING.

Project Felkay/G0058 **Date** 5/18/11 **Boring** B-3/SI-3

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
30–35 (32, 34)		23°	30.0'-35.0': SAMPLES FROM THE CORE BARREL; CLAYSTONE, DARK BROWN, MODERATE TO HARD, MODERATE TO STRONG, FINELY LAMINATED, WELL CEMENTED.	RUN 7				HQ	10:31AM 0/5 27mins	-NO CIRCULATION, NO RECOVERY?! PULLED THE CORE BARREL OUT OF THE HOLE AND THE END WAS PLUGGED W/ HARD PIECES OF CLAYSTONE. -PUSHED CASING TO 20' PUT ON CARBIDE BIT
35–38.5 (36, 38)		HARDER	35.0'-38.0': CLAYSTONE, DARK BROWN, MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, INTENSELY FRACTURED, FINELY LAMINATED.	RUN 8 BOX 1				HQ	10:58AM 12:16PM 3.5/4.0' 13mins 12:29PM	-STILL LOSING RETURN FLOW.
38.5–40		HARD	@38.5: 1.5" THICK, DARK GREYISH TO BLACK AND WHITE LAMINATED CLAY ZONE DIPPING ~ °; AT THE BASE OF IT, THE LAMINATIONS TRUNCATE ALONG A PAPER THIN SEAM AGAINST HARDER CLAYSTONE BELOW.	RUN 9 BOX 2				HQ	12:47PM 12:55PM	
40–45 (42, VW PIEZO, 44)		DARK BROWN; LIGHT/MED BROWN; 20° DIP; DARK BROWN	40.0'-45.0': GOOD RECOVERY. SILTY CLAYSTONE, DARK BROWN TO LAMINATED LIGHT AND MEDIUM BROWN, LOW TO MODERATE (LOCALLY) HARDNESS, WEAK TO MODERATE STRENGTH, INTENSELY FRACTURED. FEW TAR SEAMS 40'-41': LESS LAMINATED, MORE MASSIVE. FEW 1/4" CLASTS.	RUN 10 BOX 2				HQ	1:06PM 1:12PM 1:36PM 5.0/5.0' 13mins	-LOSING FLUID RETURN. @43: VIBRATING WIRE PIEZOMETER SENSOR S/N 1107480
45–50 (46, 48)		SOFT; SOFT	45.0'-50.0': SILTY TO FINE SANDY CLAYSTONE; MEDIUM BROWN, FINELY LAMINATED, MODERATE TO HARD, MODERATE STRENGTH, CLOSELY FRACTURED, TAR SEAMS ALONG FRACTURES, MODERATELY CEMENTED.	RUN 11 BOX 2				HQ	1:43PM 1:59PM 4.0/5.0' 20mins	NO FLUID RETURN. -DRILL RATE INCREASED A COUPLE TIMES AT THE START OF THE RUN THEN SLOWED.
50–55 (52, 54)		TAR SEAMS; BUFF TO TAN; 30° DIP; MED BRN	50.0'-55.0': WELL CEMENTED SHALE, BUFF TO TAN CHANGING TO MEDIUM BROWN @ 53.25', HARD TO VERY HARD, STRONG TO VERY STRONG, INTENSELY FRACTURED W/ TAR SEAMS RUNNING ALONG FRACTURES, SOME QUARTZ FISSURES, FINELY LAMINATED, CONTORTED BEDS @ 53 (SOFT SEDIMENT DEFORMATION), SILICA CEMENTED.	RUN 12 BOX 3; RUN 13 BOX 3				HQ; HQ	2:19PM 2:48PM 2:45PM 3.0/4.0' 4:24PM	-SWITCHED TO DIAMOND BIT. NO FLUID RETURN
55–60 (56, 58)		TAR SEAMS; 21° DIP	55.0'-60.0': CEMENTED SHALE, MEDIUM TO DARK BROWN, MODERATE TO HARD, MODERATE TO STRONG, CLOSELY FRACTURED, LOCALLY SILICA CEMENTED, FINELY LAMINATED, NUMEROUS FRACTURES FILLED W/ TAR SEAMS	RUN 14 Box 3				HQ	4:56PM 5.0/5.0 25mins	LITTLE TO NO FLUID RETURN DURING CORING RUN. ·PUSHED CASING DOWN TO 25'
60–65 (60, 62)		20° DIP	60.0'-65.0': SILTY CLAYSTONE; MEDIUM TO DARK BROWN LAMINATIONS, MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, NOT AS WELL CEMENTED AS ABOVE, CLOSELY FRACTURED, FEW TAR SEAMS ALONG FRACTURES, FEW 1/4"-1/2" CLASTS/PEBBLES ALONG LAMINATIONS.	RUN 15				HQ	5:21PM 6:13PM 5.0/5.0' 30mins	GOOD RETURN FLOW, NO FLUID LOSS

Project Felkay/G0058 **Date** 5/18/11 **Boring** B-3/SI-3

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
		20° DIP		RUN 15 BOX 4				HQ	5/5' 6:43PM	END OF DAY
66, 68		HARD	65.0'-70.0': SILTY CLAYSTONE; WELL CEMENTED W/ SILICA, LAMINATED DARK BROWN W/ LOCALLY WHITE TO TAN CHERTY LAYERS (66'), MODERATE TO HARD, MODERATE TO STRONG, INTENSELY TO CLOSELY FRACTURED, CONTORTED BEDS FROM 65.5'-67.0' (SOFT SEDIMENT DEFORMATION), FEW 1/4" CLASTS AND TAR SEAMS ALONG FRACTURES.	RUN 16 BOX 4				HQ	8:12AM 5.0'/5.0' 21 mins	7:30AM 5/19/11 -66- -GOOD CIRCULATION -NO FLUID LOSS -68-
70		BRECCIA TAR SEAMS	70.0'-71.0': SILTY CLAYSTONE; WELL CEMENTED, SIMILAR TO ABOVE, ABUNDANT TAR SEAMS AND FRACTURES, FEW OFFSET LAMINATIONS.						8:33AM 8:50AM	-70-
72		MASSIVE LOW HD WEAK	71.0'-74.0': CLAYSTONE, DARK BROWN, LOW HARDNESS, WEAK, MASSIVE, H_2S ODOR, UNIFORM COLOR AND COMPOSITION. LITTLE FRACTURING	RUN 17 BOX 4/5				HQ	5.0'/5.0'	-GOOD CIRCULATION -NO FLUID LOSS -72-
74		20° DIP HARD	74.0'-75.0': SILTY CLAYSTONE; FINELY LAMINATED MEDIUM TO DARK BROWN, MODERATE HARDNESS AND STRENGTH, MODERATELY CEMENTED.						23 mins	-74-
		HARD	75.0'-76.0': SIMILAR TO ABOVE. MEDIUM BROWN TO BUFF						9:13AM 9:37AM	
76		WEAK D. BRN	76.0'-77.0': CLAYSTONE; LAMINATED DARK BROWN W/ WHITE STRIPES, WEAK, LOW HARDNESS, CRUSHED, FRIABLE.	RUN 18 BOX 5				HQ	4.4'/5.0'	-76- -GOOD CIRCULATION
78			77.0'-80.0': TRANSITION BACK TO SILTY CLAYSTONE, LAMINATED BUFF TO TAN AND MEDIUM BROWN, MODERATE HARDNESS/STRENGTH, INTENSELY FRACTURED, SOME SILICA CEMENT.						23 mins	-78-
80, 82, 84		CRUSHED	80.0'-85.0': CORE IS VERY BROKEN UP. CLAYSTONE, MEDIUM TO DARK BROWN LAMINATED, FRIABLE TO LOW HARDNESS, WEAK, CRUSHED, STRONG H_2S ODOR.	RUN 19 BOX 5/6				HQ	10:00AM 10:16AM 3.0/5.0' 24 mins	-80- -GOOD CIRCULATION -82- -84-
86, 88			85.0'-90.0': CLAYSTONE; DARK BROWN W/ NUMEROUS WHITE TO TAN ELONGATE CLASTS ALIGNED W/ THE LAMINATIONS, LOW TO MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, CRUSHED TO INTENSELY FRACTURED, STRONG H_2S ODOR	RUN 20 BOX 6				HQ	10:41AM 11:01AM 5.0'/5.0' 26 mins	-86- GOOD CIRCULATION -88-
90			90.0'-92.0': CLAYSTONE, SIMILAR TO ABOVE, CRUSHED TO INTENSELY FRACTURED.						11:27AM 11:49AM	-90- GOOD CIRCULATION
92 VW PIEZO, 94		20° DIP HARD TAR SEAMS	92.0'-95.0': SILTY CLAYSTONE W/ FINE SAND; MEDIUM TO DARK BROWN, FINELY LAMINATED, MODERATELY HARD/STRONG, CLOSELY FRACTURED, FEW TAR SEAMS ALONG FRACTURES, WELL CEMENTED.	RUN 21 BOX 6/7				HQ	4.0'/5.0' 16 mins	-92- @93: VIBRATING WIRE PIEZOMETER SENSOR S/N 1107565 -94-
96			95.0'-99.0': SILTY CLAYSTONE W/ FINE SAND, DARK BROWN, MODERATELY HARD/STRONG, LOCALLY WEAK, FINELY LAMINATED, WELL CEMENTED	RUN 22 BOX 7				HQ	12:05PM 12:25PM 2.0'/4.0' 22 mins	-96-

Project Felkay/G0058 Date 5/19/11 Boring B-3/SI-3

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
		SANDY							12:47PM	
			TD = 99'							1:50PM: PULLING OUT HQ CORE DRILL RODS
100			7:30PM: INSTALLED 100' OF 2.75"Ø SLOPE INDICATOR SLOPE INCLINOMETER QC CASING WITH TWO GEOKON VIBRATING WIRE PIEZOMETERS TAPED TO THE SIDE OF THE CASING AT 45' AND 95' BGS, 2' OF STICK UP.							
102			5/20/11, 8:00AM: GROUTED THE BORING WITH BENTONITE/CEMENT PER MIX:							
104			30 GALS H_2O/94 lb CEMENT/25 lb BENTONITE.							
106										
108										
110										
112										
114										
116										
118										
120										
122										
124										
126										
128										
130										

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Felkay **Boring** B-4/SI-4

Location 1921 ECDLL; ABOVE HEADSCARP **Project No.** G0058

Drilling Contractor/Rig Britton Exploration/ **Date of Drilling** 5/20/11

Ground Surface Elev. ~124' **Logged By** JD **Hole Diameter** 6"ϕ HOLLOW STEM AUGERS,

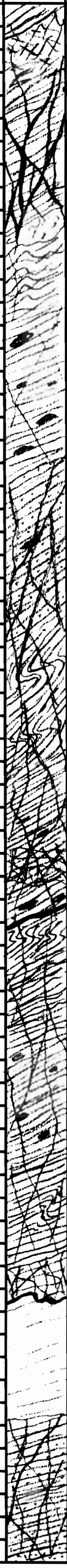
Surface BARE SOIL **Weather** FOGGY, COOL, BREEZY

ARTIFICIAL FILL (0–2.75')

WEATHERERED MONTEREY FORMATION (2.75'–30')

Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
0–2	AF	0.0'-2.75': ARTIFICIAL FILL; SILTY CLAY W/ SAND, DAMP, FIRM TO STIFF, MODERATE PLASTICITY.					HA		10:35AM: START DRILLING W/ HOLLOW STEM AUGERS. DRILLER: PAUL HELPER: SERGIO
2–4		2.75'-100.00': MONTEREY FORMATION; AS DESCRIBED BELOW 2.75'-3.5': DEEPLY WEATHERED CLAYSTONE; WHITE TO TAN W/ YELLOW ORANGE STAINING, SOFT, MOIST, PLASTIC, FIRM	1			6 11 15	MC	12"/18"	
4–5							HA		
5–6.5		5.0'-6.5': DEEPLY WEATHERED CLAYSTONE; WHITE TO TAN WITH YELLOW ORANGE STAINING, MOIST, VERY PLASTIC, SOIL-LIKE, FIRM TO STIFF, SOFT, FRIABLE,	2			22 14 17	MC	14"/18	10:50AM
6.5–8							HA		
8–9.5		8.0'-9.5': SIMILAR TO ABOVE WITH HARDER FRAGMENTS OF CLAYSTONE, LITTLE MOISTURE ON SAMPLER AND THE SAMPLE TUBES, BUT THE CLAYSTONE IS JUST MOIST.	3			17 14 26	MC	14"/18"	11:05AM
9.5–15							HA		
15–16.5		15.0'-16.0': DEEPLY WEATHERED CLAYSTONE; WHITE TO TAN W/ EXTENSIVE YELLOW ORANGE IRON OXIDE STAINING, MOIST, VERY PLASTIC, SOIL-LIKE, STIFF, SOFT TO LOCALLY HARD, FRIABLE AND WEAK, FINELY LAMINATED, BLACK TAR SEAMS, SOME SILICA CEMENTED CLAYSTONE PIECES	5 4			13 33 27	MC	14"/18"	11:20AM
16.5–20							HA		
20–21.5	CLAYEY	20.0'-21.5': SIMILAR TO ABOVE. LAST 4" OF SAMPLE (IN SHOE) IS VERY PLASTIC, STIFF CLAYSTONE, MOIST, SOIL-LIKE, DEEPLY WEATHERED, SOFT.	6			15 5 7	MC	14"/18"	11:34AM SWITCHING TO HQ CORING SYSTEM
21.5–25	HARD CLAYEY	21.5-23.4': SILICEOUS SHALE; WHITE TO TAN FINELY LAMINATED, HARD, STRONG. 23.4'-25.0': CLAYSTONE; GREYISH BROWN MOTTLED W/ YELLOW ORANGE, SOIL-LIKE, VERY STIFF, VERY PLASTIC, SOFT TO LOW HARDNESS, PLASTIC STRENGTH, MOIST, VERY CLAYEY, LAMINATIONS NOT APPARENT. DEEPLY WEATHERED.	RUN 1 BOX 1				HQ	1:54PM 2.2/3.5 6mins 2:00PM	-SPORADIC RETURN FLOW
25–30	DARK BROWN	25.0'-27.5': SILICEOUS SHALE; TAN TO BUFF, HARD, STRONG, LAMINATED. 27.5'-30.0': CLAYSTONE; DARK BROWN.	RUN 2 BOX 1				HQ	2:17PM 1.0/5.0 17mins 2:34PM	-SPORADIC RETURN FLOW. -FLUIDS TURN DARK BROWN THROUGH MIDDLE OF RUN

Project Felkay/G0058 Date 5/20/11 Boring B-4/SI-4



Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
30–35 (32, 34)	BRECCIA; 45° DIP; TAN/BUFF; TAR SEAMS; CHERTY	30.0'-35.0': SILICEOUS SHALE; MEDIUM TO DARK BROWN TRANSITIONING TO BUFF TO TAN @ 32', DAMP, HARD TO VERY HARD, STRONG TO VERY STRONG, CRUSHED TO INTENSELY FRACTURED, NUMEROUS BRECCIA ZONE w/ SILICA CEMENTING THE BRECCIATED PIECES TOGETHER, FINELY LAMINATED, ABUNDANT TAR SEAMS ALONG FRACTURES.	RUN 3 Box 1				HQ	2:47PM 5.0'/5.0' 29 mins 3:15PM	GOOD RETURN FLOW AND CIRCULATION
35–40 (36, 38)	CHERTY; BROWN	35.0'-40.0': SILICEOUS SHALE; BUFF TO TAN TRANSITIONING TO DARK BROWN @ 36', DAMP, HARD TO VERY HARD, STRONG, INTENSELY FRACTURED, BRECIA ZONE FROM 35'-36', FINELY LAMINATED, FEW TAR SEAMS, FEW ELONGATE CLASTS ALONG BEDDING	RUN 4 Box 1/2				HQ	3:34PM 5.0'/5.0' 30 mins 4:04PM	
40–45 (40, 42, 44)	33° DIP; 35° DIP; TAR SEAMS	40.0'-45.0': CLAYSTONE; DARK BROWN, MODERATE TO HARD, MODERATELY STRONG, CLOSELY FRACTURED, FINELY LAMINATED, TAR SEAMS ALONG FRACTURES, DISRUPTED/FOLDED BEDDING @ 44' MODERATELY WELL CEMENTED. SILTY TO SLIGHTLY FINE SANDY, FEW CLASTS ALONG BEDDING	RUN 5 Box 2				HQ	7:43AM 5.0'/5.0' 19 mins	END OF DAY 5/23/11, 7:30AM GOOD CIRCULATION AND RETURN FLOW
45–50 (46, 48, 50)	40° DIP; 28° DIP; 28° DIP; TAR SEAMS; WEAK; CLAY FILM	45.0'-50.0': CLAYSTONE; DARK BROWN, MODERATE TO HARD (LOCALLY LOW), MODERATELY STRONG (LOCALLY WEAK), CLOSELY FRACTURED (LOCALLY CRUSHED), FINELY LAMINATED w/ DISRUPTED/FOLDED BEDDING @ 46', MODERATELY WELL CEMENTED, TAR FILLED FRACTURES, FEW CLASTS 48.25-49.25: WEAKER, CRUSHED, CLAYSTONE. @ 49.75: PAPER THIN CLAY FILM, MUSTARD YELLOW GREEN, BRITTLE, PLASTIC, NOT SHEARED.	RUN 6 Box 3				HQ	8:02AM 8:16AM 5.0'/5.0' 29 mins 8:45AM	
50–55 (52, 54)	CHERTY; CHERTY; 30° DIP; 28° DIP	50.0'-55.0': CLAYSTONE; DARK BROWN, MODERATE TO HARD, MODERATELY STRONG, CLOSELY FRACTURED w/ TAR SEAMS ALONG FRACTURES, SILICA CEMENTED w/ CHERTY ZONES AT 51' AND 52', FINELY LAMINATED w/ DISRUPTED/FOLDED BEDDING @ 50.5', FEW CLASTS ALONG BEDDED NEAR END OF RUN.	RUN 7 Box 3/4				HQ	9:00AM 5.0'/5.0' 20 mins	
55–60 (56, 58, 60)	CRUSHED; CLAY FILM; MASSIVE; LOW HARD	55.0'-57.5': CLAYSTONE; DARK BROWN, MODERATE TO HARD, MODERATELY STRONG, CLOSELY FRACTURED, SILICA CEMENTED, FINELY LAMINATED, FAULTED/FOLDED BEDS AT 56', FEW TAR FILLED FRACTURES. 57.5'-58.25': CRUSHED CLAYSTONE, DARK BROWN, WEAK, LOW HARDNESS. CLAYEY FILM ALONG UNEVEN SURFACE @ 58.25', PAPER THIN, MUSTARD YELLOW GREEN. 58.25'-60.0': MASSIVE CLAYSTONE; DARK BROWN, LOW HARDNESS, WEAK TO MODERATE STRENGTH, LAMINATIONS FAINT TO NONEXISTENT, H_2S ODOR	RUN 8 Box 4				HQ	9:20AM 9:38AM 5.0'/5.0' 27 mins 10:05AM	
60–65 (62)	MASSIVE LOW HARD; HARDER; SOFTER; HARDER	60.0-60.9': MASSIVE CLAYSTONE; SAME AS ABOVE. 60.9'-65.0': SILTY CLAYSTONE; TAN TO MEDIUM BROWN, LOW TO MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, FINELY LAMINATED, SLIGHTLY FINE SANDY, INTENSELY TO CLOSELY FRACTURED, ABUNDANT AND LOCALLY THICK TAR SEAMS ALONG FRACTURES.	RUN 9 Box 4/5				HQ	10:28AM 4.8'/5.0' 18 mins	

UNOXIDIZED MONTEREY FORMATION

Cotton, Shires & Associates, Inc.
CONSULTING ENGINEERS AND GEOLOGISTS

Project Felkay/G0058 Date 5/23/11 Boring B-4/SI-4

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
	TAR SEAMS		RUN 9						GOOD RETURN FLOW/ CIRCULATION
			BOX 5					10:46AM	
66	CLAY FILM	65.0'-70.0': SILTY CLAYSTONE; ALTERNATING LAMINATIONS OF BUFF/TAN AND MEDIUM AND DARK BROWN, LOW TO MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, CLOSELY FRACTURED. H_2S ODOR.	RUN 10				HQ	11:03AM 4.5'/5.0'	66
68	CLASTS 25° DIP	@ 67': 1/8" THICK CLAY FILM, MUSTARD YELLOW GREEN, SOFT, WET, PLASTIC, NO APPARENT SHEARING.	BOX 5					17mins	68
70	CLAY FILM	70.0'-75.0': CLAYSTONE w/ BEDDING PARALLEL CLASTS, DARK BROWN w/ TAN/BUFF LAMINATIONS AND/OR CLASTS, LOW TO MODERATE HARD, WEAK TO MODERATE STRENGTH, CLOSELY TO MODERATELY FRACTURED, STRONG H_2S ODOR. @ 70.4': BEDDING PARALLEL PAPER TO 1/8" THICK CLAY SEAMS, NOT SHEARED BUT VERY PLASTIC AND SLICK, MUSTARD YELLOW GREEN						11:20AM 11:37AM	70
72	GRAY CLAY FILM 25° DIP		RUN 11				HQ	5.0'/5.0'	72
74	NO CLASTS 28° DIP		BOX 5/6					17mins	74
		75.0'-77.0': SIMILAR TO ABOVE.						11:54AM 12:14PM	
76		77.0-80.0': WELL CEMENTED CLAYSTONE; DARK BROWN, LAMINATED, MODERATELY HARD, MODERATELY STRONG, INTENSELY FRACTURED w/ TAR SEAMS, SILICA CEMENTED, FEW 1/2" TO 1/4" CLASTS, SILTY TO FINE SANDY.	RUN 12				HQ	5.0'/5.0'	76
78	HARD 25° DIP CRUSHED HARD	@ 79': CRUSHED ZONE, LESS CEMENTED CLAYSTONE	BOX 6					17mins	78
80	WEAK	80.0'-85.0': SILTY CLAYSTONE; DARK BROWN, LAMINATED, MODERATELY HARD/STRONG, INTENSELY TO CLOSELY FRACTURED, SILICA CEMENTED, FEW CLASTS, SOME FINE SAND.						12:31PM 12:46PM	80
82		@ 81.5', 84': WEAKER CLAYSTONE, CRUSHED.	RUN 13				HQ	4.4'/5.0'	82
84	WEAK	@ 83.0-83.5': TECTONICALLY FOLDED/FAULTED: SLICKENSIDED SURFACE, SUB HORIZONTAL, WAVY, SHINY, NO CLAY.	BOX 6/7					19mins	84
86		85.0'-87.0': CLAYSTONE; DARK BROWN, LOW HARDNESS, WEAK STRENGTH, INTENSELY FRACTURED FISSILE, BRITTLE	RUN 14					1:05PM 1:21PM	86
88	MED YEL BRN	87.0-90.0': TRANSITIONS TO SANDY SILTY CLAYSTONE, MEDIUM YELLOW BROWN, LOW HARDNESS, WEAK STRENGTH, DAMP, FRIABLE, CRUMBLY, FINE SAND, STRONG H_2S ODOR. BEDDING NOT VERY APPARENT.	BOX 7				HQ	4.4'/5.0' 18mins	88
90	MED YEL BRN	90.0'-92.5': SIMILAR TO ABOVE. TRANSITIONS FROM MEDIUM YELLOW BROWN TO TANNISH BROWN, AND LAMINATED.						1:39PM 1:57PM	90
92	HARD	92.5'-93.5': WELL CEMENTED CLAYSTONE. MEDIUM BROWN, MODERATELY HARD/STRONG, INTENSELY FRACTURED w/ NUMEROUS TAR SEAMS.	RUN 15 BOX 7				HQ	4.0'/5.0' 13mins	92
94	CRUSHED HARD	93.5'-95.0': CRUSHED CLAYSTONE TRANSITIONS TO STRIPED BUFF/TAN AND MEDIUM BROWN CLAYSTONE, LOW HARD, MODERATE STRENGTH.						2:11PM	94
	HARD	95.0'-96.0': SAME AS ABOVE.						2:30PM	
96	CRUSHED	96.0'-96.7': CRUSHED CLAYSTONE, WEAK.	RUN 16						96
		96.7'-100.0': SILTY CLAYSTONE, DARK BROWN W/ TAN LAMINATIONS, MODERATE HARD/STRONG, CLOSELY FRACTURED w/ SOME OFFSETS AND	BOX 8				HQ	4.7'/5.0' 15mins	

Project Felkay/G0058 Date 5/23/11 Boring B-4/SI-4

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
		32° DIP	TAR SEAMS, SILICA CEMENTED	RUN 16 BOX 8				HQ	4.7'/5.0' 15 mins 2:45 PM	
100			TD = 100'							100
			3:30 PM: PUMPED BENTONITE/CEMENT GROUT THROUGH THE HQ CORING RODS UP TO THE GROUND SURFACE PER MIX: 30 GALS H_2O/94 lb BAG CEMENT/25 lb BENTONITE.							
102										102
			4:30 PM: PULLED THE DRILL RODS AND CASING. SET 100' OF 2.75"ϕ SLOPE INDICATOR INCLINOMETER QC CASING.							
104										104
106										106
108										108
110										110
112										112
114										114
116										116
118										118
120										120
122										122
124										124
126										126
128										128
130										130

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project Felkay **Boring** B-5 / SI-5

Location 1921 ECDLL; DOWNSLOPE OF HEADSCARP **Project No.** G0058

Drilling Contractor/Rig Britton Exploration/ CME 550 TRACKED RIG **Date of Drilling** 5/24/11

Ground Surface Elev. ~91' **Logged By** JD **Hole Diameter** 6" HOLLOW STEM AUGER

Surface BARE SOIL **Weather** CLEAR, COOL, BREEZY

ARTIFICIAL FILL (0–4) · LANDSLIDE DEBRIS? (4–15) · WEATHERED MONTEREY FORMATION (4–27) · UNOXIDIZED MONTEREY FORMATION (27–30)

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
0–2			0.0'-4.0': ARTIFICIAL FILL; SILTY CLAY W/ ROCK FRAGMENTS, VERY PLASTIC, DARK BROWN, MOIST, FIRM, FRAGMENTS OF MONTEREY SHALE W/ TAR SEAMS					HA		DRILLER: PAUL HELPER: SERGIO 8:00AM:
2–4		HARD?	4.0'-100.0': MONTEREY FORMATION 4.0-15.0: SOIL MANTLE/DEEPLY WEATHERED BEDROCK; AS DESCRIBED BELOW.				3 4 4	MC	12"/18"	
4–6			5.0-6.5: SANDY SILT W/ CLAY; MEDIUM YELLOW BROWN, DAMP, LOOSE, FRIABLE, MORE LIKE SOIL THAN FILL, UNIFORM CONSISTENCY.	1			4 5 8	HA MC	12"/18"	HARD & GRINDING.
7–10								HA		SMOOTH & EASY
10–12			10.0-11.5: SIMILAR TO ABOVE. MEDIUM DENSE, FINE SANDY SILT, SOIL OR DEEPLY WEATHERED BEDROCK, VERY FAINT BEDROCK TEXTURE, UNIFORM COLOR & COMPOSITION, DAMP.	3 2			12 21 32	MC	16"/18"	8:35AM
13–15			15.0'-100.0': MONTEREY FORMATION; AS DESCRIBED BELOW.					HA		
15–17			15.0-16.5: SILTY CLAYSTONE; BUFF TO TAN W/ YELLOW ORANGE IRON STAINS, DAMP, WELL CEMENTED CLAYSTONE PIECES, THIN LAMINATIONS, HARD, MOD STRONG, DEEPLY WEATHERED.				4 4 5	MC	12"/18"	8:50AM
18								HA		
20–22			20.0-21.5: DEEPLY WEATHERED CLAYSTONE; DARK REDDISH BROWN, DAMP, FRIABLE, WEAK, VERY DENSE, WHITE MINERALIZATIONS ALONG LAMINATIONS, THIN TAR SEAMS.				28 50/2" –	MC	16"/18"	9:03AM
23–24								HA		
25–27		WEAK	25.0'-26.5: WEATHERED CLAYSTONE; DARK BROWN, DAMP, LOW HARDNESS, WEAK, LAMINATED, FEW CLASTS ALONG BEDDING.	4			50/5.5" – –	MC	10"/18"	9:25AM PULLING AUGERS SETTING UP TO CORE W/ HQ SYSTEM. DIAMOND BIT
27–30		25° DIP TAR SEAMS 25° DIP	25.0-30.0: SILTY CLAYSTONE; DARK BROWN, DAMP, MODERATE TO HARD, MODERATE STRENGTH, INTENSELY FRACTURED (LOCALLY CRUSHED), NUMEROUS OFFSETS ALONG TAR FILLED FRACTURES, NUMEROUS BEDDING ALIGNED CLASTS, LITTLE TO NO WEATHERING.	RUN 1 BOX 1				HQ	12:07PM 5.0'/5.0' 26mins 12:33PM	11:15AM CORED TO 25'. CASING DOWN TO 25' GOOD FLUID CIRCULATION

Project Felkay/G0058 Date 5/24/11 Boring B-5/SI-5

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
32, 34		25° DIP; CLAY FILM; 25° DIP	30.0'-35.0': SILTY CLAYSTONE w/ SOME FINE SAND, LAMINATED MEDIUM GREY BROWN AND DARK BROWN w/ 1/2" TO 1" ELONGATE CLASTS ALONG BEDDING, MODERATE TO HARD, MODERATE TO STRONG, INTENSELY TO CLOSELY FRACTURED w/ TAR SEAMS ALONG FRACTURES, SILICA CEMENTED. @ 32.5': 1/8" THICK MUSTARD YELLOW GREEN PLASTIC CLAY FILM, NO SHEARS APPARENT, BUT SLICK.	RUN 2 BOX 1/2				HQ	12:47PM 5.0'/5.0' 20mins 1:07PM	GOOD FLUID CIRCULATION
36		HARD	35.0'-36.5': SIMILAR TO ABOVE, HARD, STRONG, SILICA CEMENTED						1:21PM	
38		DRK BRN	36.5-39.0': CLAYSTONE; LOW HARDNESS, WEAK STRENGTH, MEDIUM TO DARK BROWN w/ WHITE TO TAN LAMINATIONS/STRINGERS, CLOSELY FRACTURED, BREAKS ALONG BEDDING.	RUN 3 BOX 2				HQ	5.0'/5.0'	
		CRUSHED	39.0'-39.5': CRUSHED CLAYSTONE, MEDIUM GREY BROWN.						15mins	
40		HARD; HARD	39.5'-40.0': WELL CEMENTED CLAYSTONE, HARD, STRONG, LAMINATED, TAR SEAMS						1:36PM 1:52PM	
			40.0'-41.0'- SAME AS ABOVE.							
42		DRK BRN MASSIVE	41.0'-43.0': MASSIVE CLAYSTONE; DARK BROWN, LOW HARDNESS, WEAK TO MODERATE STRENGTH, LITTLE FRACTURING, LAMINATIONS NOT APPARENT, STRONG H₂S ODOR.	RUN 4 BOX 2/3				HQ	4.9'/5.0'	
44		CRUSHED; HARD; OIL	43.5'-45.0': SILTY CLAYSTONE w/ FINE SAND, MEDIUM BROWN w/ GRAY BROWN INTERBEDS, MODERATE HARDNESS/STRENGTH, INTENSELY FRACTURED w/ TAR SEAMS ALONG FRACTURES.						27mins 2:19PM	
		HARD	45.0'-46.0': SAME AS ABOVE, OIL SHOW ON SAMPLE						2:34PM	
46, 48		20° DIP; DRK BRN HARD; 20° DIP	46.0'-49.0': SILTY CLAYSTONE; MEDIUM BROWN, FINELY LAMINATED, WELL CEMENTED w/ SILICA, MODERATE TO HARD, MODERATE TO STRONG, TRANSITIONS TO BUFF TO TAN, LAMINATED SHALE, HARD, STRONG, CLOSELY FRACTURED, TAR FILLED FRACTURES.	RUN 5 BOX 3				HQ	5.0'/5.0' 20mins	
50		BUFF/TAN HARD; BUFF/TAN	50.0'-50.8': BUFF/TAN SILICEOUS SHALE AS ABOVE.						2:54PM 3:18PM	
52, 54		BRECCIA TAR FILM 26° DIP; 28° DIP; TAR SEAMS	50.8'-55.0': SILTY CLAYSTONE; MEDIUM BROWN TO GREY BROWN LAMINATIONS, MODERATE TO HARD, MODERATE TO STRONG, MODERATELY SILICA CEMENTED, NUMEROUS TAR FILLED FRACTURES, INTENSELY FRACTURES. @ 52': 1" THICK BRECCIA OR GRAVEL LAG ZONE, SOFT SEDIMENT DEFORMATION?	RUN 6 BOX 3/4				HQ	5.0'/5.0' 29mins	
56, 58, 60		30° DIP; 30° DIP	55.0'-60.0': THE ENTIRE CORE IS INTACT. 55.0'-56.0': INTENSELY FRACTURED. SILTY CLAYSTONE; MEDIUM BROWN AND GREY BROWN LAMINATIONS, MODERATE TO STRONG, MODERATE TO HARD, CLOSELY FRACTURED w/ FEW TAR SEAMS, FEW CLASTS ALONG BEDDING, MODERATELY SILICA CEMENTED.	RUN 7 BOX 4				HQ	3:47PM 4:02PM 5.0'/5.0' 18mins 4:20PM	
62		28° DIP; WEAK CRUSHED	60.0'-65.0': SILTY CLAYSTONE; LAMINATED MEDIUM BROWN AND GREY BROWN, MODERATELY HARD, MODERATELY STRONG, CLOSELY FRACTURED w/ FEW TAR SEAMS, FEW BEDDING ALIGNED CLASTS, MODERATELY SILICA CEMENTED. 61.5'-63.0': CLAYSTONE; DARK BROWN, LOW HARDNESS, WEAK STRENGTH.	RUN 8 BOX 4/5				HQ	4:35PM 5.0'/5.0' 38mins	

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Project Felkay/G0058 Date 5/24/11 Boring B-5/SI-5

UNOXIDIZED MONTEREY FORMATION

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
		25° DIP; HARD BUFF/TAN	65.0'-66.0': SILICEOUS SHALE; BUFF TO TAN, HARD TO VERY HARD, STRONG, FINELY LAMINATED, INTENSELY FRACTURED.	RUN 8 BOX 5				HQ	38mins 5:13PM 5:28PM	GOOD FLUID CIRCULATION
66			66.0'-70.0': TRANSITIONS TO DARK BROWN, SILICEOUS SHALE, HARD, STRONG, LOCALLY CONTORTED LAMINATIONS @ 66', INTENSELY FRACTURED.	RUN 9 BOX 5				HQ	4.8'/5.0'	66
68		MED BRN HARD							26mins	68
			70.0'-71.4': SIMILAR TO ABOVE. CLOSELY FRACTURED.						5:54PM	
70		DK BRN HARD							7:51AM	70 END OF DAY 5/25/11, 7:30AM
		20° DIP	71.4'-74.6': CLAYSTONE; DARK BROWN, MORE MASSIVE, LAMINATIONS ARE NOT APPARENT, LOW HARDNESS, WEAK STRENGTH, CLOSELY TO MODERATELY FRACTURED, UNIFORM COLOR AND COMPOSITION. H_2S ODOR	RUN 10 BOX 5/6				HQ	5.0'/5.0'	GOOD FLUID CIRCULATION
72		DK BRN MASSIVE								72
74			74.6'-76.0': SILTY CLAYSTONE; BUFF TO TAN AND MEDIUM BROWN LAMINATIONS, MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, CLOSELY FRACTURED.						40mins 8:31AM 8:49AM	74
76		DK BRN	76.0'-77.0': CLAYSTONE; DARK BROWN W/ TAN CLASTS/STRINGERS ALONG LAMINATIONS, LOW HARDNESS, WEAK STRENGTH.	RUN 11				HQ	5.0'/5.0'	76
78		HARDER; 22° DIP	77.0'-80.0': SILTY CLAYSTONE; BUFF TO TAN AND MEDIUM BROWN LAMINATIONS, MODERATE HARDNESS/STRENGTH, LOCALLY WEAK, CLOSELY FRACTURED. FEW TAR FILLED FRACTURES.	BOX 6					20mins 9:09AM	78
80							X		9:25AM	80
82		CLAY FILM	80.0'-85.0': SILTY CLAYSTONE; MEDIUM BROWN AND DARK BROWN LAMINATIONS W/ TAN CLASTS AND STRINGERS ALIGNED W/ BEDDING, LOW TO MODERATE HARDNESS, WEAK TO MODERATE STRENGTH, DAMP, CLOSELY FRACTURED, FEW TAR FILLED FRACTURES.	RUN 12 BOX 7				HQ	4.2'/5.0' 17mins	82
84									9:42AM	84
86		CLAY FILM	85.0'-90.0': CLAYSTONE; MEDIUM BROWN TRANSITIONING TO DARK BROWN W/ TAN CLASTS AND STRINGERS ALIGNE W/ BEDDING, LOW HARDNESS, WEAK, INTENSELY FRACTURE, STRONG H_2S ODOR.	RUN 13				HQ	9:59AM 5.0'/5.0'	86
88			@ 85.5': THIN GRAYISH PLASTIC CLAY FILM.	BOX 7					26mins	88
			90.0'-90.8': SIMILAR TO ABOVE; LOW HARDNESS, WEAK						10:25AM	
90		20° DIP HARD	90.8'-93.5': SILTY CLAYSTONE; MEDIUM TO DARK BROWN, FINELY LAMINATED, WELL CEMENTED, MODERATELY HARD/STRONG, TAR FILLED FRACTURES, INTENSELY TO CLOSELY FRACTURED.						10:41AM	90
92		CLAY FILM	@ 93.5': 1/8" THICK MUSTARD YELLOW GREEN CLAY SEAM, INCLINED ALONG BEDDING, STIFF, PLASTIC, NOT SHEARED, VERY SLICK. * GOOD SAMPLE	RUN 14 BOX 8				HQ	5.0'/5.0'	92
* 94		HARD; 20° DIP; HARD.	93.5'-95.0': SIMILAR TO ABOVE; VARIABLE HARDNESS AND STRENGTH.						21mins 11:02AM	94
96		<10° DIP; CLAY FILM; HARD	95.0'-98.5': SILTY CLAYSTONE; DARK BROWN, VARIABLY CEMENTED, MODERATE TO HARD, MODERATE TO STRONG, CLOSELY FRACTURED, FEW TAR FILLED FRACTURES, NUMEROUS TAN CLASTS/STRINGERS @ 97': 1/3" THICK CLAY SEAM, MUSTARD YELLOW GREEN, STIFF, UNSHEARED.	RUN 15 BOX 8				HQ	11:20AM 5.0'/5.0' 20mins	96

Project Felkay/G0058 **Date** 5/25/11 **Boring** B-5/SI-5

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
SHEARS		10° DIP SANDY	98.5'-100.0': FINE SANDY SILT w/ CLAY; MEDIUM YELLOW BROWN, MODERATE HARD/STRONG, MODERATELY WELL CEMENTED, STEEPLY DIPS LAMINATIONS, TECTONICALLY STRIATED SHEARS @ 98.8'-98.9'	RUN 15 BOX 8				HQ	5.0'/5.0' 20 mins 11:40AM	
100			TD = 100'							100
			1:05PM: PUMPED ~100 GALS OF BENTONITE/CEMENT GROUT THROUGH THE DRILL RODS PER MIX:							
102			30 GALS H_2O/94 lb BAG CEMENT/25 lb BENTONITE							102
104			2:15PM: PULLED ALL THE DRILL RODS AND CASING OUT OF THE HOLE AND WET SET 100' OF 2.75"Ø SLOPE INDICATOR QC INCLINOMETER CASING w/ ~0.5' STICKUP							104
106										106
108										108
110										110
112										112
114										114
116										116
118										118
120										120
122										122
124										124
126										126
128										128
130										130

COTTON, SHIRES AND ASSOCIATES, INC.

LOG OF EXPLORATORY DRILLING

Project Felkay **Boring** CSA/LD-2

Location 1925 ECDLL, 6' WEST OF CSA/B-2 **Project No.** G0058

Drilling Contractor/Rig RC DRILLING/LIMITED ACCESS RIG **Date of Drilling** 10/11/11

Ground Surface Elev. ______ **Logged By** SD **Hole Diameter** 24" Ø AUGER

Surface BARE SOIL **Weather** CLEAR, WARM, BREEZY

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
0										12:00PM: START DRILLING W/ 24" Ø AUGER.
2										DRILLER: RICHARD HELPER: RICH
4			: MONTEREY FORMATION; DEEPLY WEATHERED CLAYSTONE AND SHALE, MEDIUM YELLOW ORANGE, YELLOW BROWN TO DARK BROWN, DAMP TO MOIST, LOW HARDNESS, PLASTIC TO WEAK STRENGTH, HIGHLY FRACTURED, THINNLY LAMINATED.							
7			7.0 - 10.0: DARK BROWN CLAYSTONE W/ HARDER WELLCEMENTED SHALE, MOIST TO WET, PLASTIC TO MODERATE STRENGTH.							
10			10.0 - 12.0: MED. YELLOW ORN CLYST, MOIST TO WET, PLASTIC TO LOW HARDNESS, WEAK							12:15PM: @ 10.5'
14			@ 14.5: MED YEL BRN/YEL ORN CLYST, MOIST, SLIGHT SMELL OF H_2S,							12:24PM: @ 14.5'
16										12:30PM: @ 16' 12:30PM - 1:00PM: LUNCH 1:10PM: DRILLING AGAIN
18			@ 18.5: SPOILS ARE MOIST TO ALMOST WET. MEDIUM TAN TO BROWN CLAYSTONE, VERY PLASTIC, LOW HARDNESS, WEAK STRENGTH.							1:20PM: @ 18.5'
20		HARD	@ 20.0: HARDER DRILLING							1:26PM: 20.0' HARDER - DRILLING [illegible]
22			@ 22.5: DARK REDDISH BROWN CLAYSTONE W/ LARGER FRAGMENTS, WET							1:33PM: @ 22.5'
23		▽	@ 23: FREE WATER ON SPOILS							
25			@ 25.0: WATER STANDING ON BOTTOM OF HOLE							1:42PM: @ 25'
26			@ 26.5: HARDER							1:56PM: @ 26.5' SWITCH TO 12" Ø CORE BARREL. 4:05PM: BOH @ 27' WATER @ 23'

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project: Felkay
Boring: CSA/LD-1
Location: 1921 ECDL / 8' EAST OF CSA/B-4 (PADRE DH-3)
Project No.: G0058
Drilling Contractor/Rig:
Date of Drilling: 10/10/11
Ground Surface Elev.:
Logged By: SS/JD
Hole Diameter: ~24"ø
Surface:
Weather:

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
0–2			0.0'~27.0': LARGE-DIAMETER BORING CUTTINGS COMPOSED OF MONTEREY FORMATION CLAYSTONE/SILTSTONE.							
2–3			Till From PADRE Boring 6							2
4										4
6										6
8										8
10										10
12										12
14										14
16										16
18										18
19										@19' 11:30AM: JL LOGGED [illegible]
20										20
22										22
24										24
25										@26' WATER IN BOTTOM OF [illegible]
26										26
>27' ([illegible] OF PADRE DH-3)										11:52AM: [illegible]
28										28 [illegible]
28.5										12:14PM: ~38.5
29										12:31PM - 1:00PM: DRILLER TAKE LUNCH.

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Sheet 1 of

COTTON, SHIRES AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project FOXCAY
Boring LD-1
Location 1921 ECDLL | 8 FT EAST OF CSA B-4 (THROUGH PADRE DH-3)
Project No. G0058
Drilling Contractor/Rig RC DRILLING | LAR.
Date of Drilling 10/10/11
Ground Surface Elev.
Logged By SS/JD
Hole Diameter 24 INCHES
Surface DIRT
Weather EARLY AM FOG - CLEARING

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov. (%)	Remarks
			FILL (DH-3 BACKFILL) SANDY CLAY; YEL BRN, MD PLASTIC, MOIST W/ ANG. GRAVEL TO FINE COBBLE CLASTS IN MONTEREY SHALE							MOB 7:30-10:22 AM BEGIN DRILLING 10:23 AM THROUGH PADRE DH-3 SPOILS 10:37 AM - 6 FT 10:42 AM - 11 FT 10:54 - 13 FT 11:07 - 19 FT ROCK? BOULDER? SWITCH TO 18" CORE BARREL

Sheet 1 of

428.1019'

COTTON, SHIRES AND ASSOCIATES, INC.

LOG OF EXPLORATORY DRILLING

Project: FELKAY	Boring: LD-3
Location: 1921 ECDLL / UPSLOPE OF CSA / B-3	Project No. G0058
Drilling Contractor/Rig: RC DRILLING / LIMITED ACCESS RIG	Date of Drilling: 10/13/11
Ground Surface Elev. ±92' — Logged By: SS	Hole Diameter: 2 FT
Surface: BARE SOIL	Weather: CLEAR, HOT

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Blows/ft	Sample Type	Recov.	Remarks
0–12.4 (LANDSLIDE DEPOSITS — REMNANT MONTEREY FORMATION)			**0 - 12.4 FT - LANDSLIDE DEPOSITS (REMNANT MONTEREY FORMATION)**						SAMPLE 12 FT	BEGIN 9:12 AM WITH AUGER
2			0 - 0.3 FT - SOIL FORMATION SILTY CLAY, YELLBRN, DRY, CRUMBLY, ABUNDANT ROOTLETS + ROOTHOLES; SUBHORIZONTAL CONTACT TO		FRACTURES IN UPPER ARE GENERALLY NEAR VERTICAL & CLOSED TO 1 MM					@3': BN67W, 30SW
4			0.3 FT - 12.4 FT - SHALE; LT GRY - YELLOW BRN, - BLK, THINLY BEDDED, SILICEOUS, INTENSELY FRACTURED, WEAK TO MODERATE HARDNESS, ABUNDANT ROOTLETS							@4.5': BN70W, 29SW; 9:37 AM
6				B-1				B	X	
8			@4.5 FT - 0.5 FT THICK CLAYSTONE; LT OLIVE-ORANGE BRN, STIFF, MOIST, PLASTIC; TRACE ROOTLETS BELOW 4.5 FT							9:44 AM; @6.9': BN59W, 30SW
10			@5.4 FT - 0.5 FT THICK CLAYSTONE; LT OLIVE-ORANGE BRN, STIFF, MOIST, PLASTIC							@10': BN53W, 26SW
12			@6.7 - 6.9 FT - CLAYSTONE; OLIVE BRN, STIFF, MOIST, PLASTIC	BS-1; BS-2					X; X	10:06 AM; @11.3': BN61W, 25SW; DRILLER BEGINS USING DOWNPRESSURE
14			@7.35 - 7.5 FT - SILTSTONE; LT OLIVE, FRIABLE, MOIST	B-2				B	X	@12.4': CONTACT: N69W, 25SW (GOUGE ZONE); 10:23 AM
			@7.8 - 7.9 FT - SILTSTONE; LT OLIVE, FRIABLE, MOIST							@13.7': BN64W, 24SW
16			BELOW 8 FT - SHALE IS BRITTLE							10:36 AM; @16.5 FT: BN64W, 27SW
18			@11.3 FT: 0.25 INCH THICK CLAY; BRN, HIGHLY PLASTIC, STIFF, MOIST WITHIN 0.3 FT THICK SILTY CLAY; LT OLIVE, WEAK, SL MOD PLASTIC, MOIST TO SILICEOUS SHALE, THINLY BEDDED, MOD HARD, FRACTURED							@19': BN70W, 29SW
20			**@12.1 - 12.4 FT - BASAL LANDSLIDE GOUGE ZONE**							10:49 AM, 11:06 AM SWITCH TO 12 INCH CORE; 11:26 AM - ADD 2 GALLON H_2O; @21': BN69W, 25SW; 11:40 AM
22 (MONTEREY FORMATION)			0.3 FT THICK, SILTY CLAY, HIGHLY PLASTIC, OLIVE BRN, PULVERIZED, MOIST, ABUNDANT ROOTLETS, ABUNDANT ANGULAR TO TABULAR, F-C GRAVEL FRAGMENTS OF SILICEOUS SHALE ENTRAINED IN THIS ZONE WITH MANY ORIENTATIONS							12:11 AM - LUNCH TO 1 PM; 2:10 PM - SWITCH TO 24 INCH AUGER
24		SEEP	**12.4 FT - B.O.H. - MONTEREY FORMATION**							2:44 PM; ADD 1 GALLON H_2O; @24.5': BN65W, 28SW
26			SHALE; DK OLIVE BRN, MOD HARDNESS, MOIST, THINLY BEDDED, BORING IS VERY TIGHT BELOW 12.4 FT DEPTH TO BOH; SL PETROLIFEROUS ODOR BELOW 13 FT							SEEP
28			@25 - SEEP ON NE QUADRANT; @26 - SEEP ON SW QUADRANT		STANDING H_2O @ 27 FT		18:30 HR 10/13/11			3:15 PM
			TOTAL DEPTH = 30 FT; NO CAVING; SEEP @ 25 FT							3:30 PM

Sheet 1 of 1

COTTON, SHIRES & ASSOCIATES, INC.
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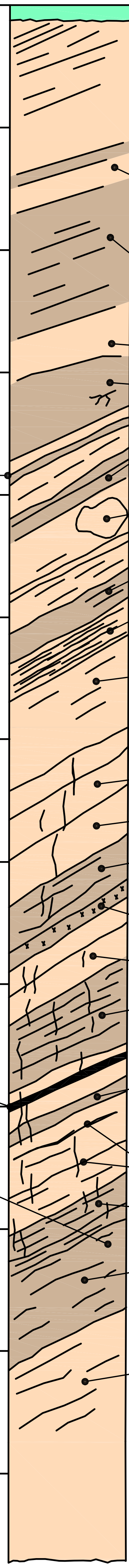
TAPE FOR DOWNHOLE LOGGING ON SW QUADRANT
SEE GS / JW FOR TOTAL DEPTH 10/14/11 & BACKFILL [BACKFILLED W/ SPOIL & TAMPED PER JW 10/14/11]

LD-1

1"=2'

S N

ELEVATION 124'



0.0'-0.2'; ARTIFICIAL FILL;

0.2'-25.4'; MONTEREY FORMATION BEDROCK;

Weathered shale and claystone; oxidized to pale orange, tan, medium brown and white; this unit is generally weak to moderately strong with localized plastic claystone interbeds, unsheared, damp to moist, closely fractured, well bedded with localized orientation changes due to folding; abundant black tar seams along fractures, fractures are tight; locally hard cemented shale near bottom of borehole.

Depth (ft)	Left annotations	Right annotations
0		
2	2.0': N10W, 31SW Bedding	
	3.0': N25W, 32SW Bedding	Average strike of bedding N60W from top of hole, Deeply weathered, cemented shale, weak, friable, damp to dry Hard and soft brittle intervals
4		Deeply weathered claystone, soil-like, moist, well bedded, Fe-stained, orange, black and gray
	5.0': N43W, 38SW Bedding	Hard brittle shale, cemented, tar seams
6		Softer, deeply weathered silty claystone, moist, white to orange, rootlets Hard, N40W, 34SW/ N30W, 33SW
	Soft, brown, chalky	Softer, low hardness, weak to moderate strength, oxidized yellow orange, moist to damp, silty clay, less soil-like
8	8.0': N65W, 34SW Bedding	Brittle, hard, strong, highly fractured Variable hardness, tan, orange to gray
	9.5': N55W, 33SW Bedding	White tan, chalky, softer
10		Well cemented, hard, brittle
		Variable hardness, well cemented to clayey, hard to soft Low to moderate hardness, tan, white, orange, moist
12		
	13.0': N25W, 31SW Bedding	White, chalky, variable hardness
		Harder, stronger shale, well cemented, gray/white
14		Brown claystone, deeply weathered, medium brown to white-tan/orange, low hardness, weak, plastic, moist, damp
		White, chalky, soft to hard (sample), moist, N50W, 33SW
		Hard, black tar seams, brittle
16	16.0': N54W, 37SW Bedding	Claystone, thin bedded, orange gray, white, moist, plastic, low hardness, weak
	Black, N45W, 33SW	Gray claystone, plastic, weak, moist gray/orange/white claystone, silty, weak white, chalky, soft plastic
18		Harder, cemented, white, chalky
	19.0': N54W, 35SW Bedding; Bag sample	Silty claystone, plastic, grayish-orange, moist, unsheared
20	20.0': N45W, 33SW Bedding	Claystone, soft, plastic, deeply weathered, highly oxidized
22		Hard, strong, shale, cemented, oxidized, white to orange, moist
24		
		Harder, stronger, well cemented shale
		Spoils pile

TD=25.4'

No groundwater encountered

Notes:
- LD-1 drilled and logged October 10, 2011
- No groundwater encountered
- Borehole video-taped October 11, 2011
- Borehole backfilled with spoils and compacted with kelly bar.
- Drilling performed by R.C. Drilling, Thousand Oaks, CA

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Large Diameter Boring LD-1

1921 El Camino De La Luz
Santa Barbara, California

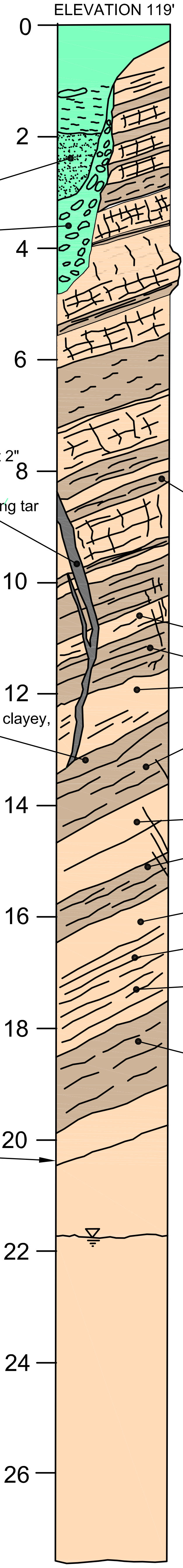
GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=2'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	B-1

LD-2

1"=2'

S N

ELEVATION 119'



TD=27.6'

0.0'-5.0'; ARTIFICIAL FILL;

Dark brown clay, moist, plastic, firm

Clean sand, fine to medium grained medium brown to brown, mottled, moist, little clay binder, medium dense, to loose

Rocky fill, brown, moist, clayey

Gray clay, offset 2"

N60E, anastamosing tar seams

10.0': N65W, 24SW Bedding

Dark brown, clayey, plastic, soft

16.0': N60W, 28SW Bedding

Lower limit of downhole logging

0.5'-27.6'; MONTEREY FORMATION BEDROCK;

Weathered shale and claystone; oxidized to pale orange, tan, medium brown and white; this unit is generally weak to moderately strong with localized plastic claystone interbeds, unsheared, damp to moist, closely fractured, well bedded with localized orientation changes due to folding; abundant black tar seams along fractures, fractures are tight; locally hard near bottom of borehole.

22° dip
Gray, tan, moist, plastic, claystone
Hard, brittle, fractures

Lots of roots
Raveling
22° dip
Gray/brown claystone, plastic, low hardness, moist
Brittle, hard, intensely fractured

Low hardness, plastic, weak, claystone, orange/brown/gray, moist, deeply weathered
Hard, strong, well cemented, white, fractured

Gray/orange claystone, low hardness, plastic
Hard, brittle
Fat, brown clay
N70W, 40SW on surface, N55W, 29SW average
White, brittle shale, hard, strong, intensely fractured
Gray with laminated claystone, plastic
Claystone, low hardness, weak, orange gray, plastic, moist

Hard, brittle, tan, buff, intensely fractured, N23E, 88NW fractures

Softer, low hardness, rootlets
Brittle, hard, well cemented, intensely fractured, strong

Softer claystone, orange/tan/light gray, moist, plastic, weak

Variable, low to moderate hardness, weak to moderate strength tan to brown

Harder, moderately strong, medium orange/tan/white
Black tar seam, fractures N75E, 70NW, N80E, 60NW
Soft, plastic

Harder, moderately strong shale

Bag sample, soft, plastic, gray/white/orange claystone

Hard, strong shale, laminated, brittle, white and tan/orange, laminations oxidized, closely fractured with tar seam filling, damp to moist

Bag sample, soft, plastic, deeply weathered claystone, white/tan orange/gray, moist, low hardness, gouges easily

Harder

Softer

Water at ~21.2'

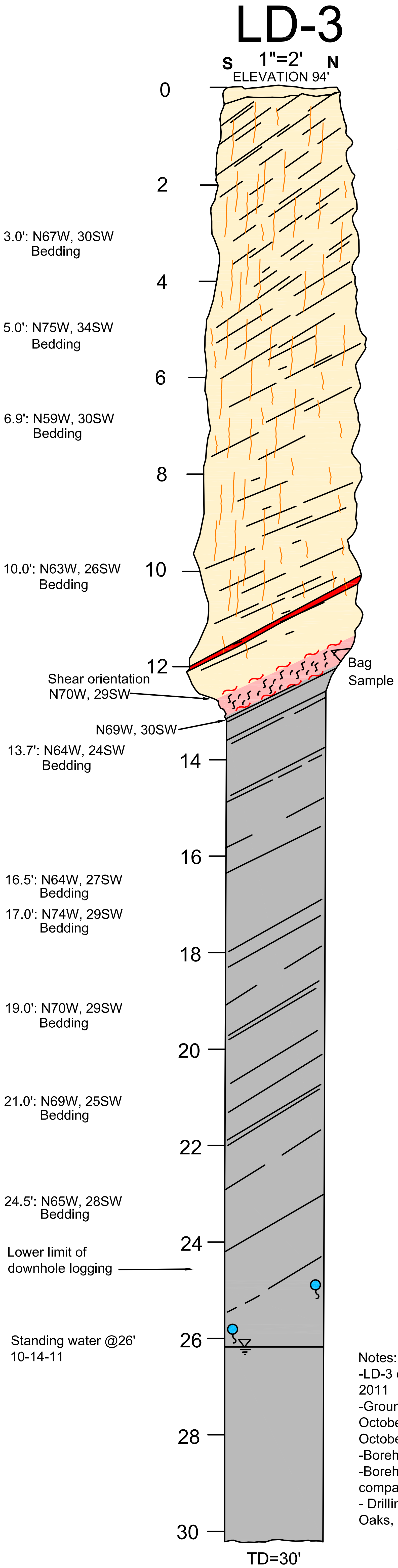
Notes:
-LD-2 drilled and logged October 11, 2011
-Groundwater encountered at 21.5'
-Borehole video-taped October 12, 2011
-Borehole backfilled with spoils and compacted with kelly bar.
- Drilling performed by R.C. Drilling, Thousand Oaks, CA

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Large Diameter Boring LD-2

1925 El Camino De La Luz
Santa Barbara, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=2'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	B-2



0.0'-12.4'; LANDSLIDE DEBRIS;

0.0'-0.3'; Silty Clay - Dark yellowish brown, dry, roots
0.3'-12.4'; DISRUPTED SHALE; Light gray to yellowish brown, thin-bedded siliceous shale, intensely to closely fractured, friable to weak strength, abundant high-angle open fractures, unstable borehole walls ravel and cave easily; shale is thinly laminated with siltstone and claystone interbeds, consistent bedding 0-12'

4.5'; 6" thick claystone, orange brown, stiff, damp, plastic, rootlets
-Near-vertical high-angle fracture zone with approximate north-south orientation along north and south walls, zone is dilated with abundant open fractures from 1/10" to 1/2", continues for most of 1-12' zone

8.0'-12.0'; Increase in brittle, siliceous shale
11.3'; 1/4" thick clay, brown, highly plastic, stiff, 1/4" thick clay is within 4" thick silty clay interbed

12.0'-12.4'; SHEAR ZONE: BASAL RUPTURE ZONE;

Pulverized shale, clay gouge, soil and roots, 3"-5" thick; shear zone is bedding plane parallel, with planar upper and lower shear boundaries with disrupted rock and soil between, 1/4" to 1/2" thick clay gouge along upper and lower shear boundaries is disrupted and discontinuous, no striae observed; loose pulverized shale between upper and lower shear boundaries, profuse root growth

12.4'-30.0'; MONTEREY FORMATION BEDROCK;

12.4'; Shale - Dark olive brown, moderately strong to strong, thin-bedded, damp, slight petroliferous odor below 13'; rock is very difficult to drill, little fracturing, unoxidized, no caving, rock is tight with consistent bedding to 24' (lower limit of logging)

- Drilling becomes very difficult at 19', using 12" core barrel to pilot drill

25.0'-26.0'; Seeps encountered during drilling

Notes:
-LD-3 drilled and logged October 13 and 14, 2011
-Groundwater first encountered ar 25' on October 13 during drilling, stabilized at 26' on October 14, 2011
-Borehole video-taped October 14, 2011
-Borehole backfilled with spoils and compacted with kelly bar.
- Drilling performed by R.C. Drilling, Thousand Oaks, CA

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Large Diameter Boring LD-3
1925 El Camino De La Luz
Santa Barbara, California

GEO/ENG BY	SCALE	PROJECT NO.
JW	1"=2'	G0058
APPROVED BY	DATE	FIGURE NO.
JW	OCTOBER 2012	B-3

APPENDIX B

LABORATORY TESTING

Table B-1, Summary of Laboratory Test Results
Figure B-1, Summary of Atterberg Limits
Figure B-2, Undrained, Consolidated Triaxial Compression Strength Tests
Figure B-3, Summary of Torsional Ring Shear Tests.

APPENDIX B

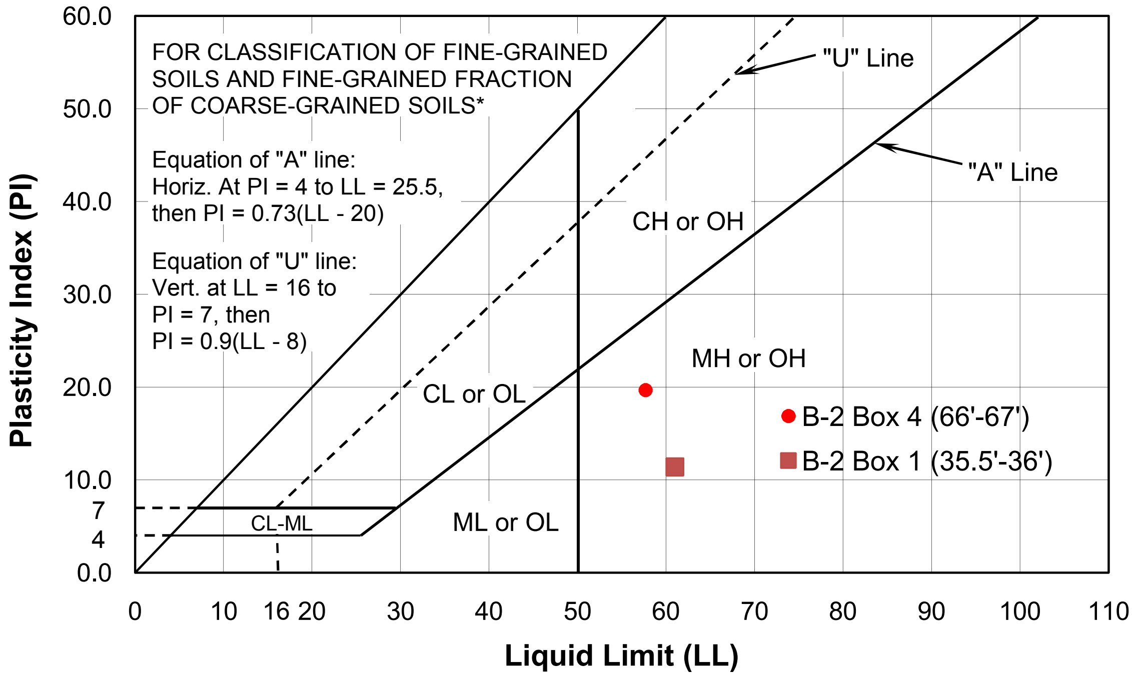
LABORATORY TESTING

The laboratory testing program performed by CSA and Cooper Testing Labs of Mountain View, California consisted of testing of the Monterey Formation claystone sampled during the field investigation to evaluate index properties and strength parameters of subsurface materials. Samples used for laboratory testing were obtained either from relatively undisturbed samples (Modified California sampling) from the upper portions of the boreholes, disturbed samples from the mud-rotary drilling (Atterberg limits testing), or bulk samples excavated from the sidewalls of the large-diameter boreholes for samples for disturbed/remolded testing. The rock descriptions and the field and laboratory test results were used to assign parameters to the various materials at the site. The results of the laboratory testing program are presented in this appendix.

The following laboratory tests were performed as part of this investigation:

1. Detailed rock description: ASTM D2487;
2. Natural moisture content of the rock: ASTM D2216;
3. In-situ unit weight of the rock (wet and dry);
4. Atterberg limits determination: ASTM D4318;
5. Unconfined compression: ASTM D2166;
6. Triaxial compression shear strength (consolidated, undrained) ASTM D4767; and,
7. Remolded torsional ring shear: ASTM D6467.

SUMMARY OF ATTERBERG LIMITS



SAMPLE DESCRIPTION	BORING No./ SAMPLE No.	DEPTH, Ft.	LIQUID LIMIT, %	PLASTICITY INDEX, %	USCS SYMBOL
Silty Claystone; Dark Brown	B-2/Box 1	35.5'-36'	61.0	11.4	MH
Claystone; Dark Brown	B-2/Box 4	66'-67'	57.7	19.7	MH

*Reference: 1995 Annual Book of ASTM Standards; ASTM Designation D4318: Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

COTTON, SHIRES, & ASSOCIATES, INC.

Atterberg Limits

Project: Felkay   Proj No. G0058   Location: 1925 ECDLL; Above Headscarp

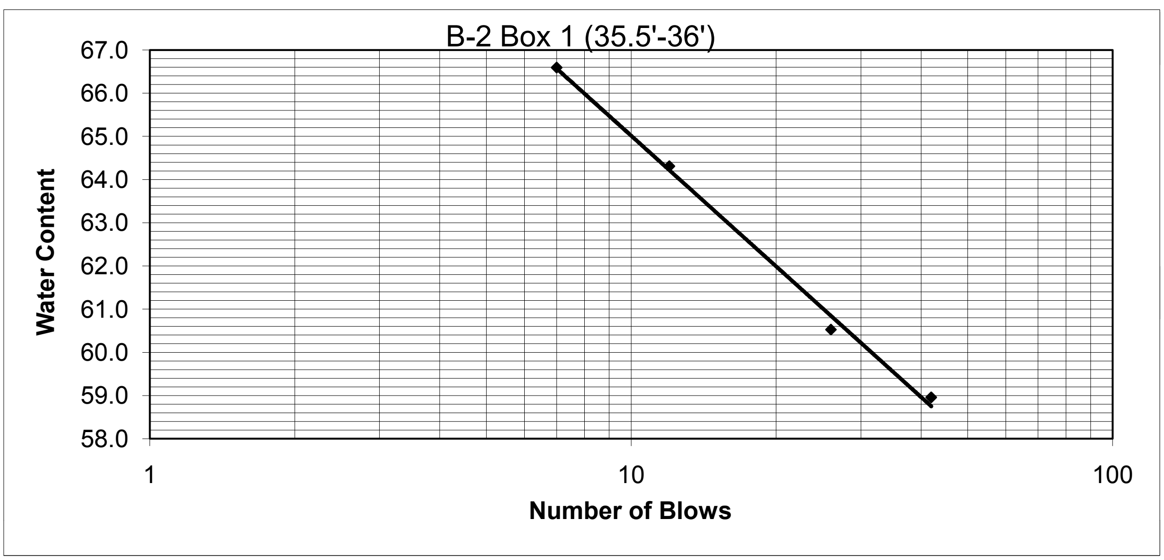
Hole: B-2
Sample: B-2 Box 1 (35.5'-36')   Description of Sample: Silty Claystone; Dark Brown

Tested by: JN   Date of Testing: 8/22/2011

	LIQUID LIMIT			
Trial Number	1	2	3	4
Tin Number	9	1	7	16
Weight of Tin	1.61	1.61	1.55	1.64
Number of Blows	7	12	26	42
Tin + Wet Soil	23.95	22.05	19.85	19.65
Tin+ Dry Soil	15.02	14.05	12.95	12.97
Weight Water	8.93	8	6.9	6.68
Weight of Dry Soil	13.41	12.44	11.4	11.33
Moisture Content (%)	66.6	64.3	60.5	59.0

	PLASTIC LIMIT			
Trial Number	1	2	3	4
Tin Number	AA-1	C	n/a	n/a
Weight of Tin	1.59	1.55		
Tin + Wet Soil	6.89	6.87		
Tin+ Dry Soil	5.13	5.11		
Weight Water	1.76	1.76		
Weight of Dry Soil	3.54	3.56		
Moisture Content (%)	49.7	49.4		

LL 61
PL 49.6
PI 11.4



Unified Soil Classification: MH

COTTON, SHIRES, & ASSOCIATES, INC.

Atterberg Limits

Project Felkay Proj No. G0058 Location 1925 ECDLL; Above Headscarp

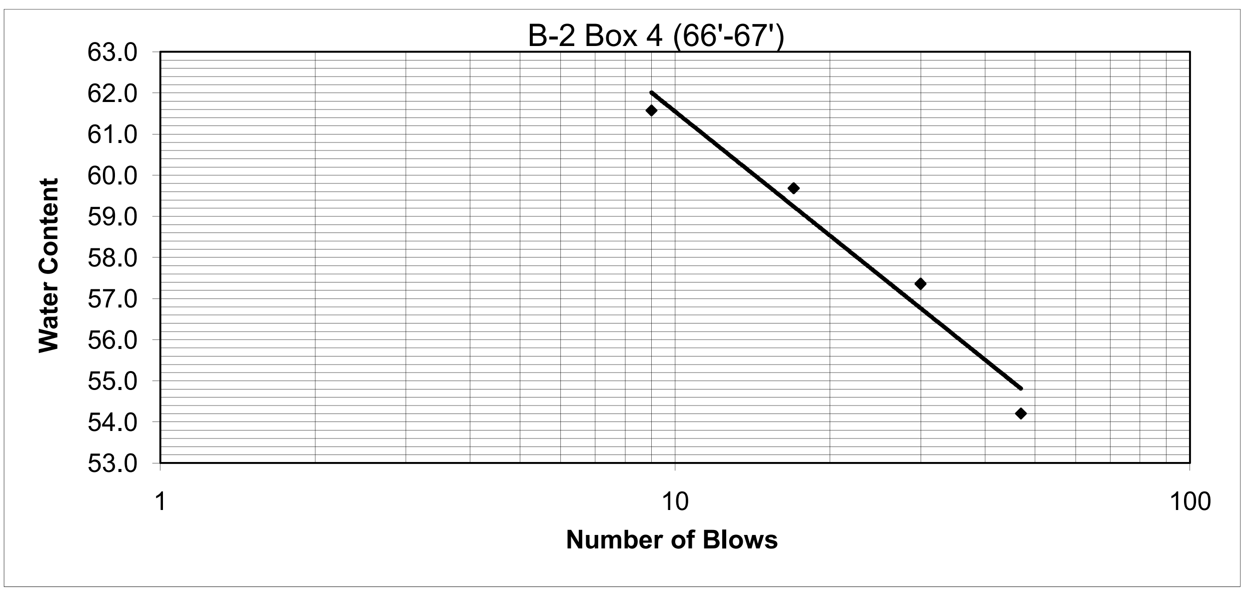
Hole B-2
Sample B-2 Box 4 (66'-67') Description of Sample Claystone; Dark Brown

Tested by JN Date of Testing 8/22/2011

	LIQUID LIMIT			
Trial Number	1	2	3	4
Tin Number	X	17	A-2	V-5
Weight of Tin	1.7	1.58	1.59	1.51
Number of Blows	9	17	30	47
Tin + Wet Soil	21.38	18.73	17.94	13.06
Tin+ Dry Soil	13.88	12.32	11.98	9
Weight Water	7.5	6.41	5.96	4.06
Weight of Dry Soil	12.18	10.74	10.39	7.49
Moisture Content (%)	61.6	59.7	57.4	54.2

	PLASTIC LIMIT			
Trial Number	1	2	3	4
Tin Number	HAA	B	n/a	n/a
Weight of Tin	1.59	1.59		
Tin + Wet Soil	5.91	7		
Tin+ Dry Soil	4.72	5.51		
Weight Water	1.19	1.49		
Weight of Dry Soil	3.13	3.92		
Moisture Content (%)	38.0	38.0		

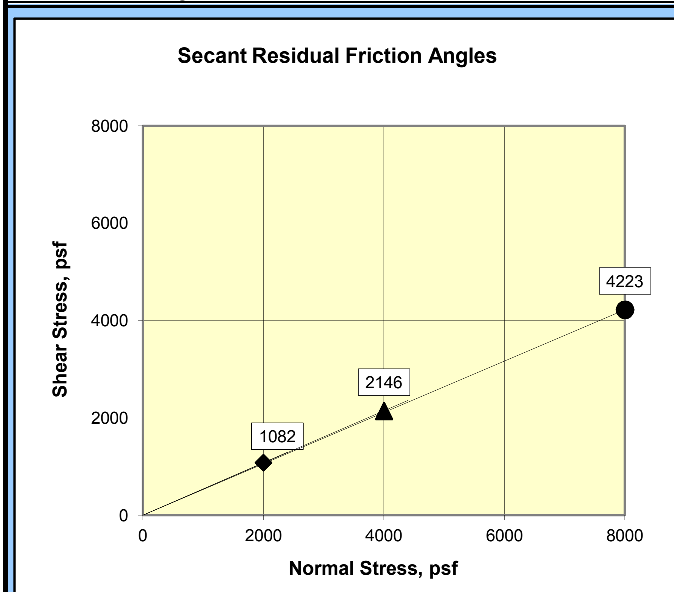
LL 57.7
PL 38.0
PI 19.7

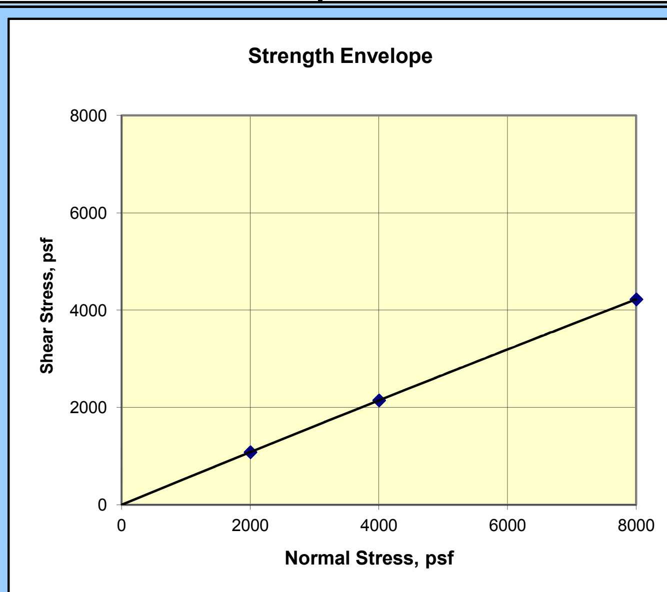


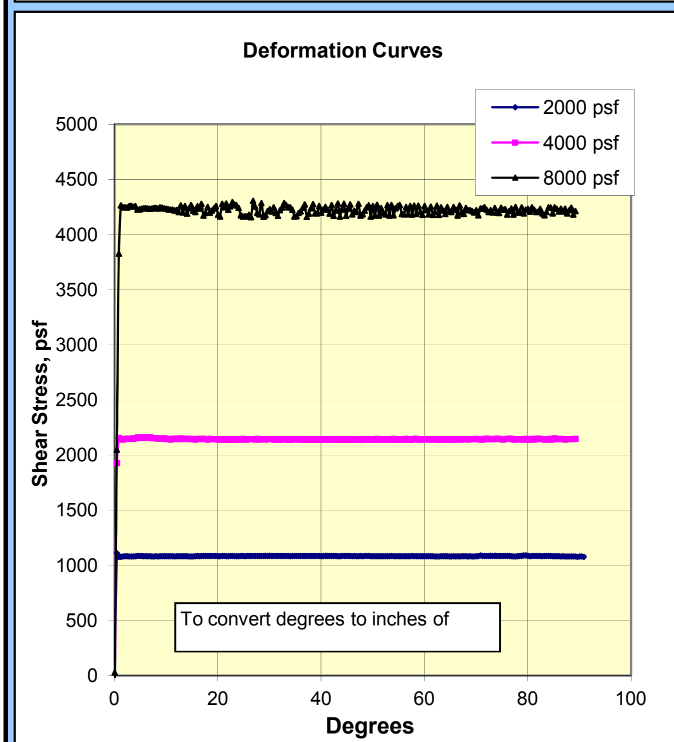
Unified Soil Classification: MH

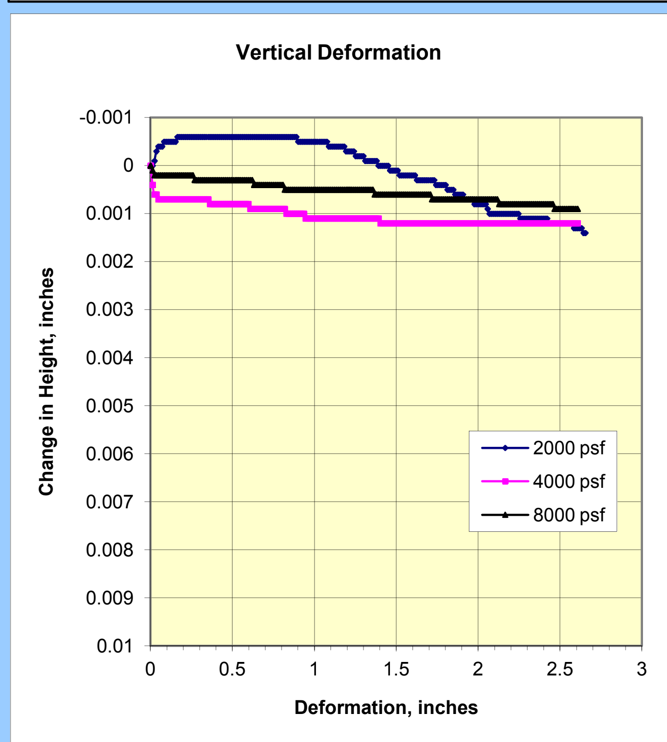
Drained, Residual Torsional Ring Shear Test ASTM D 6467

Job No.:	026-504	**Boring:**	LD-2	**Date:**	11/2/2011	**Undisturbed:**	
Client:	Cotton, Shires & Associates	**Sample:**	2	**By:**	PJ	**Peak:**	
Project:	Felkay - G0058	**Depth:**	18.0'	**Checked:**	DC	**Residual:**	
Soil Type:	Brown Elastic SILT			**Clay, %:**	39	**Fully Softened:**	X
Remarks:	A small friction correction was applied to all points.			**LL:**	61	**Peak:**	
Normal Stress, psf	2000	4000	8000	**PL:**	42	**Residual:**	X
Secant Phi, deg.	28	28	28				





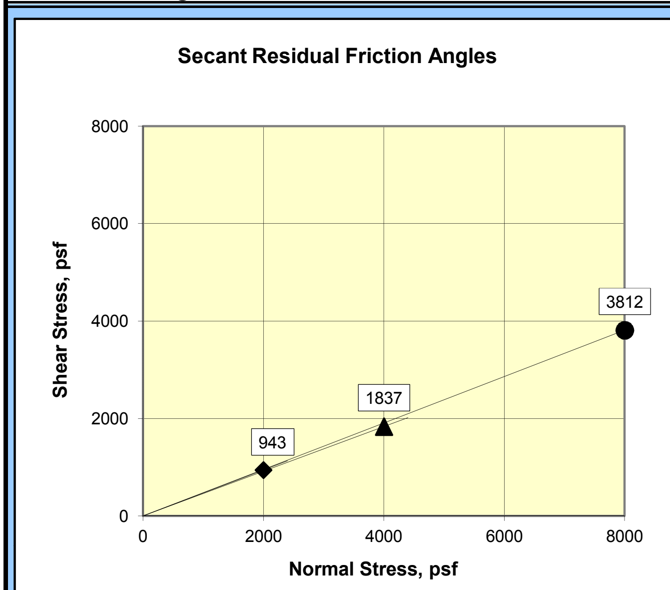


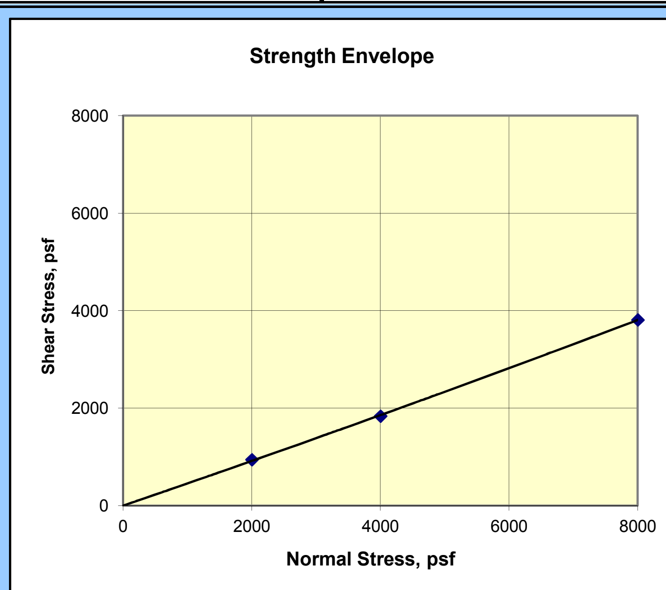


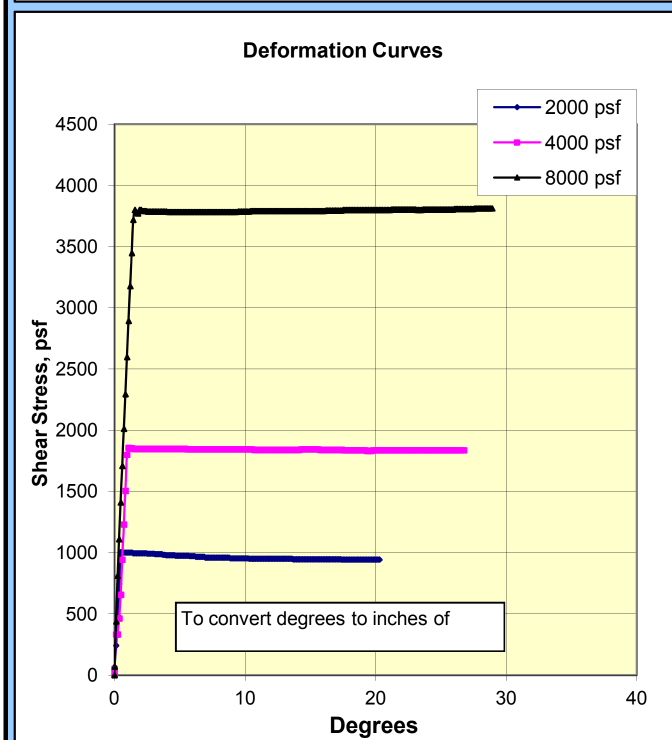
Drained, Residual Torsional Ring Shear Test ASTM D 6467

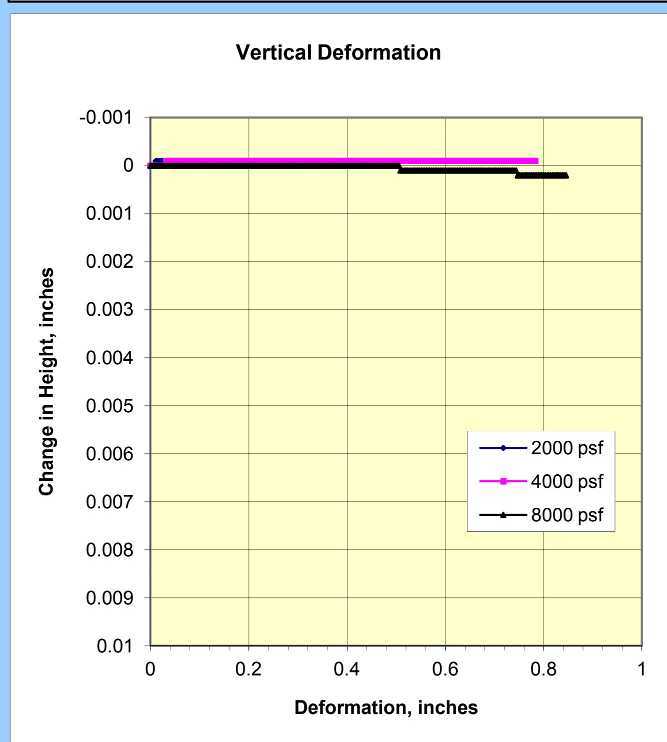
Job No.:	026-504	**Boring:**	LD-2	**Date:**	11/7/2011	**Undisturbed:**	
Client:	Cotton, Shires & Associates	**Sample:**	1	**By:**	PJ	**Peak:**	
Project:	Felkay - G0058	**Depth:**	16.5'	**Checked:**	DC	**Residual:**	
Soil Type:	Brown Elastic SILT			**Clay, %:**	39	**Fully Softened:**	X
Remarks:				**LL:**	57	**Peak:**	
				PL:	39	**Residual:**	X

Normal Stress, psf	2000	4000	8000
Secant Phi, deg.	25	25	25



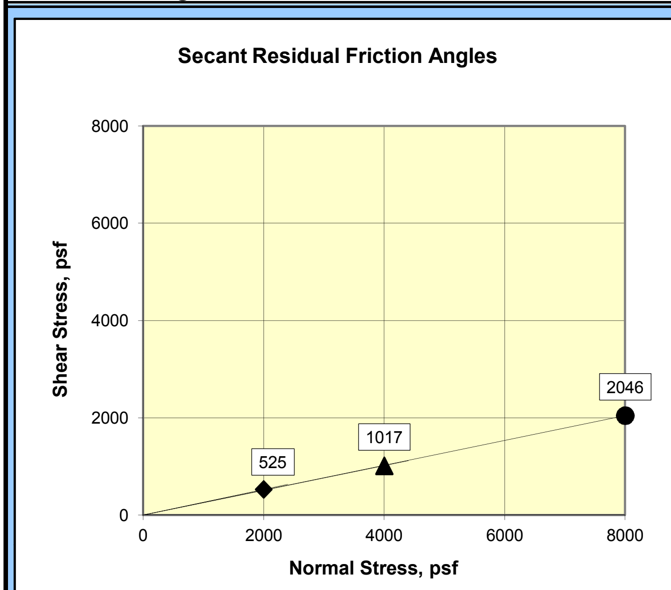


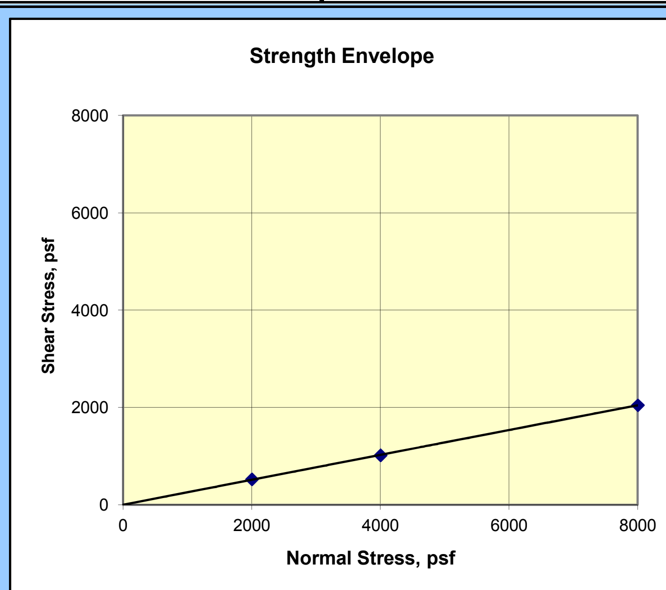


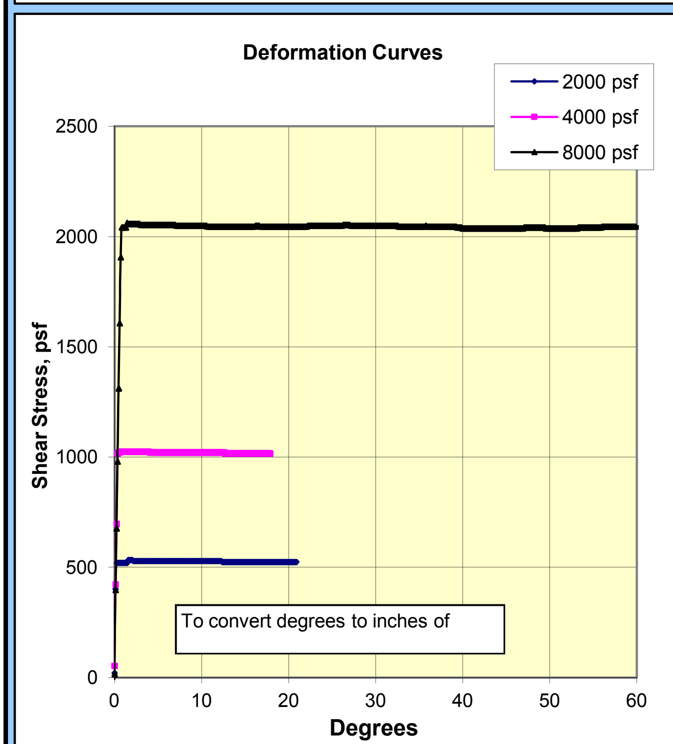


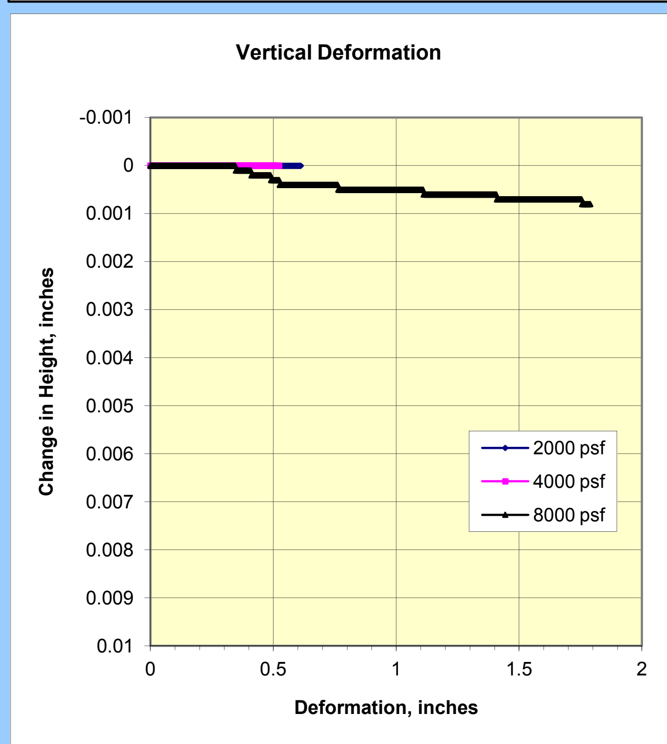
Drained, Residual Torsional Ring Shear Test ASTM D 6467

Job No.:	026-504	**Boring:**	LD-1	**Date:**	11/7/2011	**Undisturbed:**	
Client:	Cotton, Shires & Associates	**Sample:**	3a	**By:**	PJ	**Peak:**	
Project:	Felkay - G0058	**Depth:**	20.0-20.5'	**Checked:**	DC	**Residual:**	
Soil Type:	Brown Elastic SILT			**Clay, %:**	42	**Fully Softened:**	X
Remarks:	A small friction friction correction was added to each point.			**LL:**	68	**Peak:**	
Normal Stress, psf	2000 / 4000 / 8000			**PL:**	33	**Residual:**	X
Secant Phi, deg.	15 / 14 / 14						

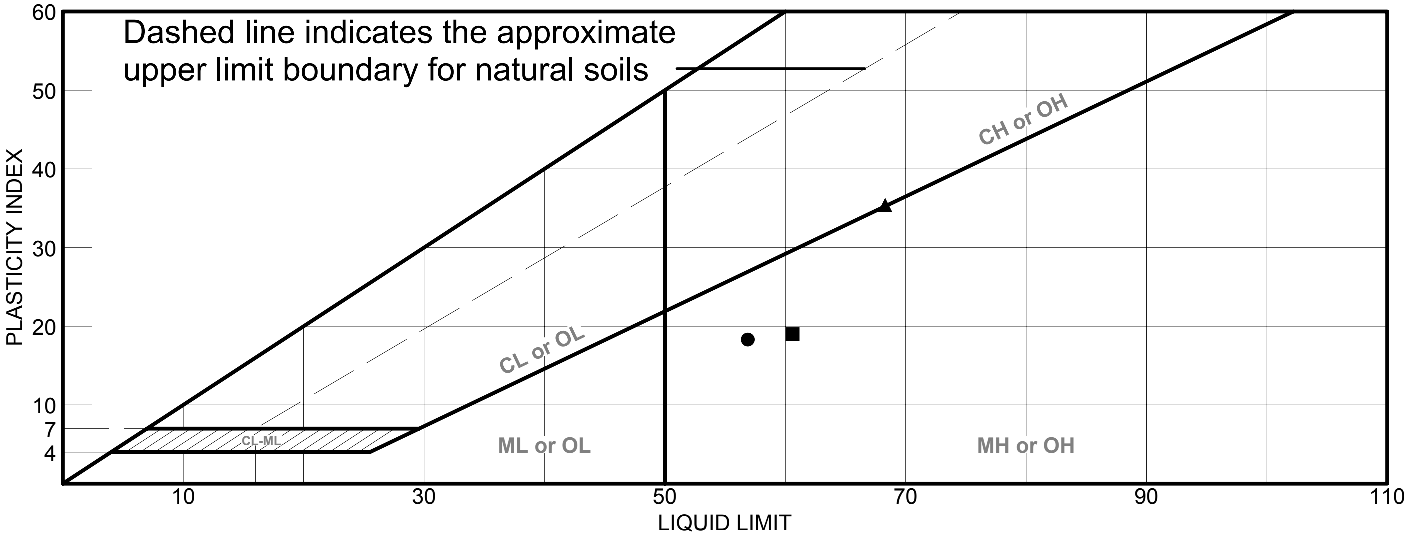


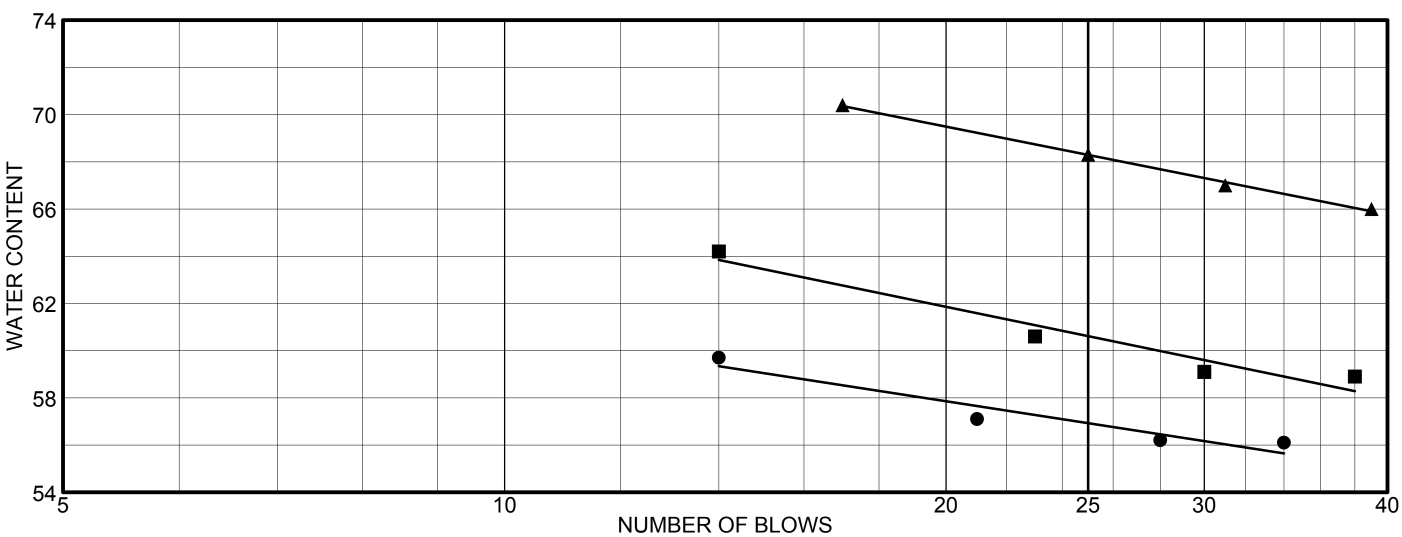






LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Elastic SILT	56.9	38.6	18.3	94.0	89.2	MH
■	Brown Elastic SILT	60.6	41.6	19.0	98.3	92.3	MH
▲	Brown Elastic SILT	68.3	32.9	35.4	99.7	98.3	MH

Project No. 026-504 **Client:** Cotton, Shires & Associates

Project: Felkay - G0058

● **Source:** LD-2 **Sample No.:** #1 **Elev./Depth:** 16.5'

■ **Source:** LD-2 **Sample No.:** #2 **Elev./Depth:** 18.0'

▲ **Source:** LD-1 **Sample No.:** #3a **Elev./Depth:** 20-20.5'

Remarks:

●

■

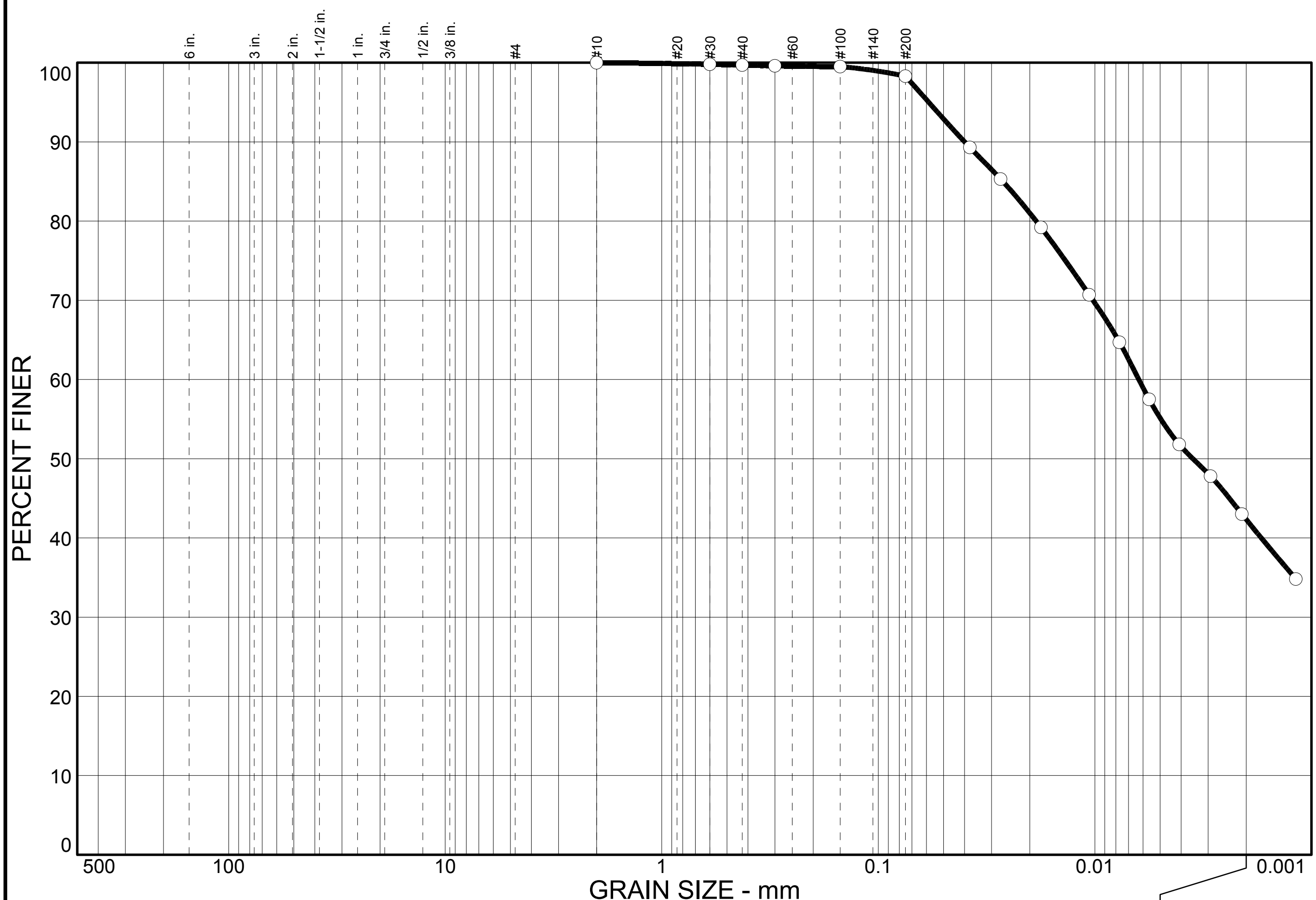
▲

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	1.7	56.0	42.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	99.8		
#40	99.7		
#50	99.6		
#100	99.5		
#200	98.3		
0.0378 mm.	89.3		
0.0272 mm.	85.3		
0.0177 mm.	79.2		
0.0106 mm.	70.7		
0.0077 mm.	64.7		
0.0056 mm.	57.5		
0.0041 mm.	51.8		
0.0029 mm.	47.8		
0.0021 mm.	43.0		
0.0012 mm.	34.8		

* (no specification provided)

Soil Description

Brown Elastic SILT

Atterberg Limits

PL= 32.9   LL= 68.3   PI= 35.4

Coefficients

D_{85}= 0.0266 D_{60}= 0.0063 D_{50}= 0.0035
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH   AASHTO=

Remarks

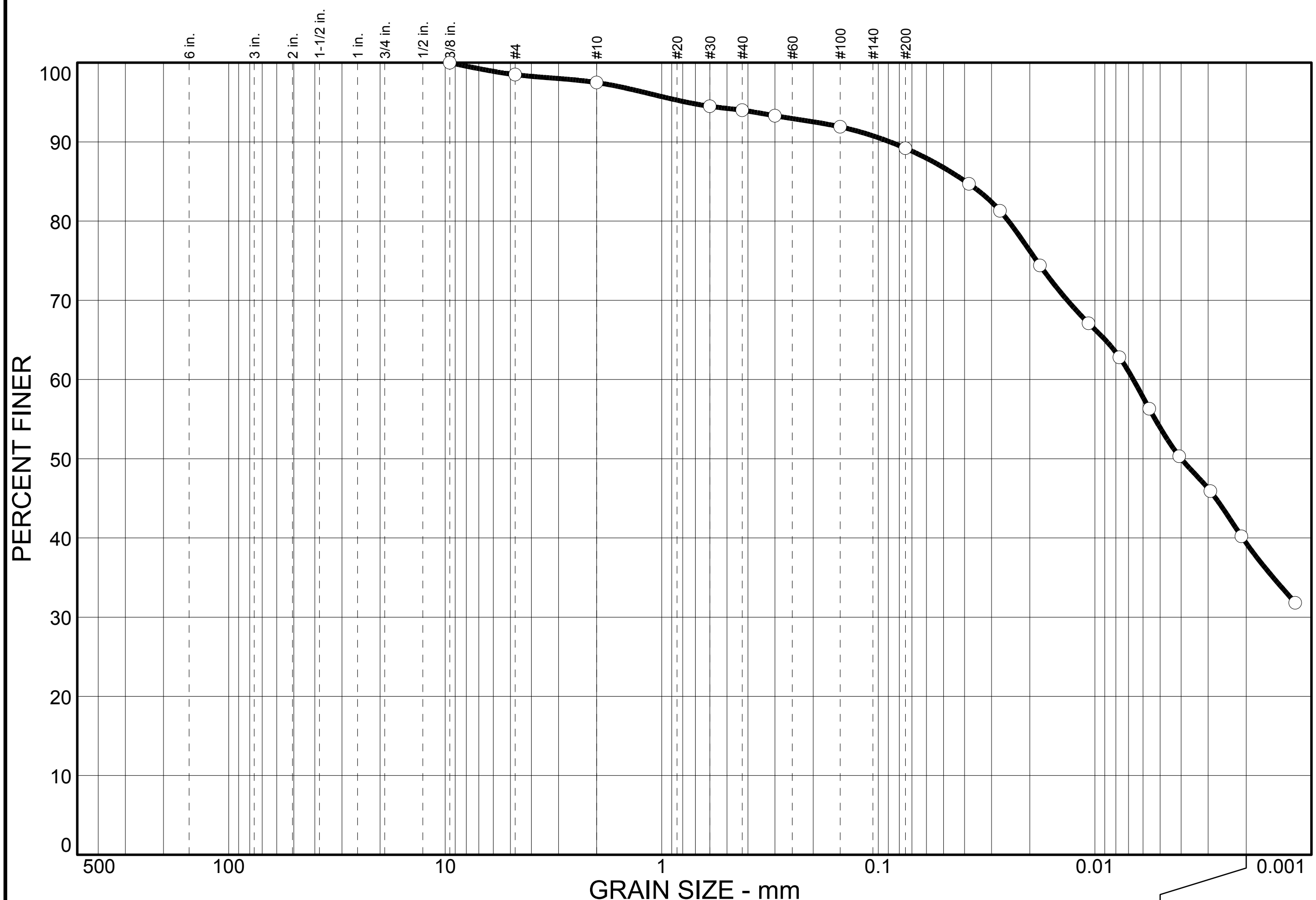
Sample No.: #3a   **Source of Sample:** LD-1   **Date:** 10/26/11
Location: **Elev./Depth:** 20-20.5'

COOPER TESTING LABORATORY

Client: Cotton, Shires & Associates
Project: Felkay - G0058

Project No: 026-504   **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.5	9.3	49.9	39.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.5		
#10	97.5		
#30	94.5		
#40	94.0		
#50	93.3		
#100	91.9		
#200	89.2		
0.0382 mm.	84.7		
0.0274 mm.	81.3		
0.0179 mm.	74.4		
0.0107 mm.	67.1		
0.0077 mm.	62.8		
0.0056 mm.	56.3		
0.0041 mm.	50.3		
0.0029 mm.	45.9		
0.0021 mm.	40.2		
0.0012 mm.	31.8		

* (no specification provided)

Soil Description

Brown Elastic SILT

Atterberg Limits

PL= 38.6 LL= 56.9 PI= 18.3

Coefficients

D_{85}= 0.0396 D_{60}= 0.0067 D_{50}= 0.0040
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

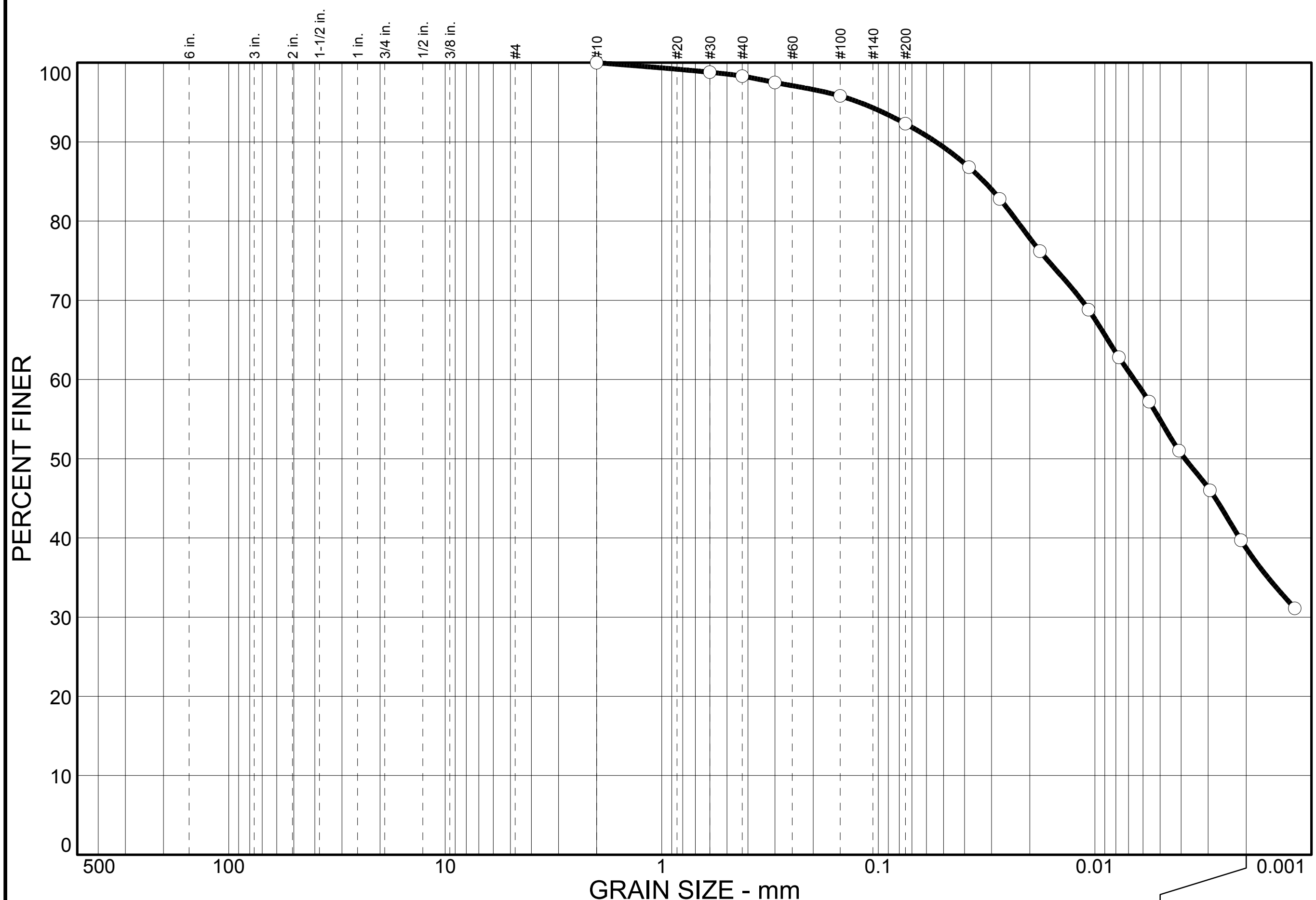
Remarks

Sample No.: #1 **Source of Sample:** LD-2 **Date:** 10/26/11
Location: **Elev./Depth:** 16.5'

COOPER TESTING LABORATORY

Client: Cotton, Shires & Associates
Project: Felkay - G0058
Project No: 026-504 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	7.7	53.6	38.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#30	98.8		
#40	98.3		
#50	97.5		
#100	95.8		
#200	92.3		
0.0382 mm.	86.8		
0.0275 mm.	82.8		
0.0179 mm.	76.2		
0.0107 mm.	68.8		
0.0077 mm.	62.8		
0.0056 mm.	57.2		
0.0041 mm.	51.0		
0.0029 mm.	46.0		
0.0021 mm.	39.7		
0.0012 mm.	31.1		

* (no specification provided)

Soil Description

Brown Elastic SILT

Atterberg Limits

PL= 41.6 LL= 60.6 PI= 19.0

Coefficients

D_{85}= 0.0325 D_{60}= 0.0066 D_{50}= 0.0038
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO=

Remarks

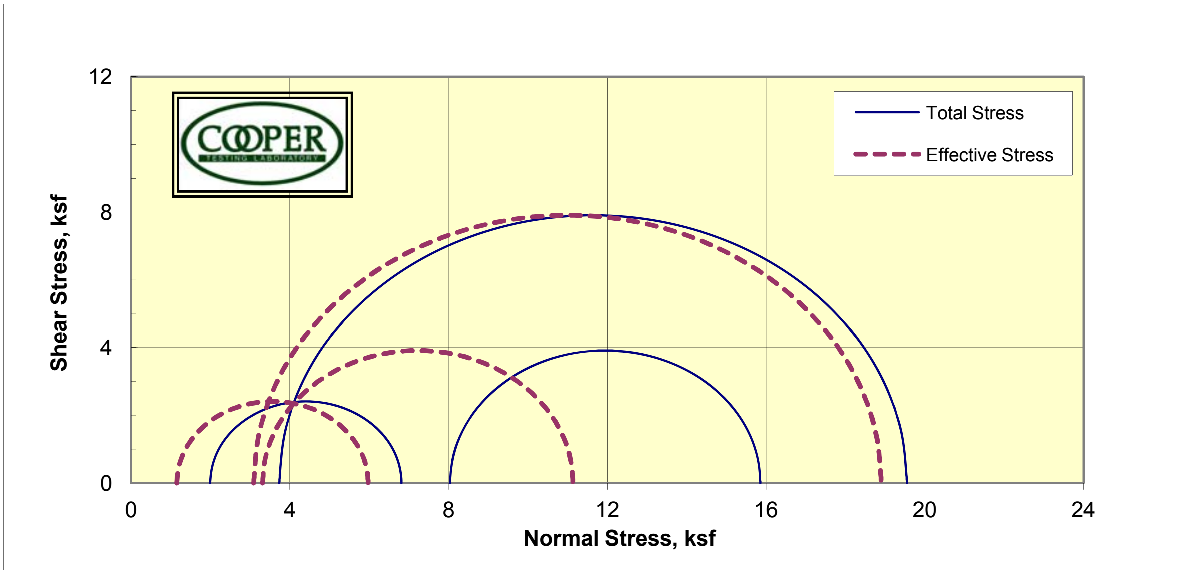
Sample No.: #2 **Source of Sample:** LD-2 **Date:** 10/26/11
Location: **Elev./Depth:** 18.0'

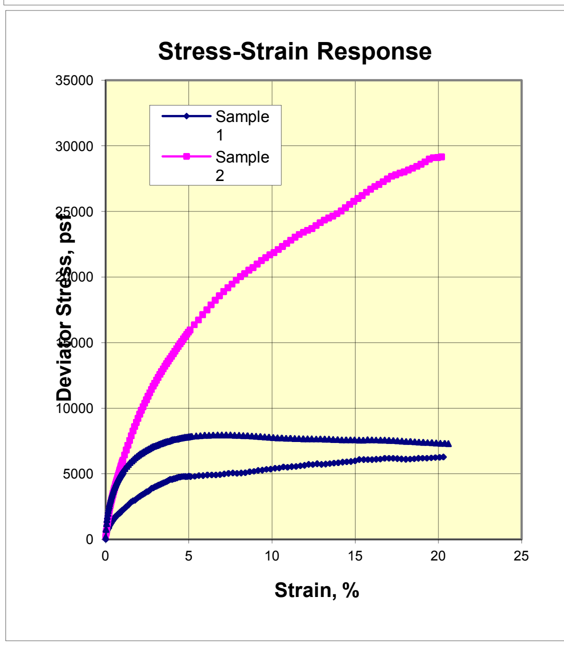
COOPER TESTING LABORATORY

Client: Cotton, Shires & Associates
Project: Felkay - G0058

Project No: 026-504 **Figure**

Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767





Sample:	1	2	3	4
MC, %	43.8	37.4	48.4	
DD, pcf	74.3	72.2	71.4	
Sat. %	93.3	75.7	95.9	
Void Ratio	1.268	1.334	1.361	
Diameter in	2.40	2.42	2.40	
Height, in	4.99	5.10	5.02	
	Final			
MC, %	43.8	40.0	45.1	
DD, pcf	77.2	81.0	76.0	
Sat. %	100.0	100.0	100.0	
Void Ratio	1.183	1.080	1.218	
Diameter, in	2.37	2.29	2.36	
Height, in	4.92	5.06	4.88	
Cell, psi	73.9	87.8	115.6	
BP, psi	60.1	61.9	59.8	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	4.820	15.813	7.823	
Excess PP	0.839	0.646	4.721	
Sigma 1	5.974	18.904	11.137	
Sigma 3	1.155	3.090	3.315	
P, ksf	3.565	10.997	7.226	
Q, ksf	2.410	7.907	3.911	
Stress Ratio	5.173	6.117	3.360	
Rate in/min	0.0005	0.0005	0.0005	

Job No.: 026-498 **Date:** 9/20/2011

Client: Cotton, Shires & Associate: BY:DC

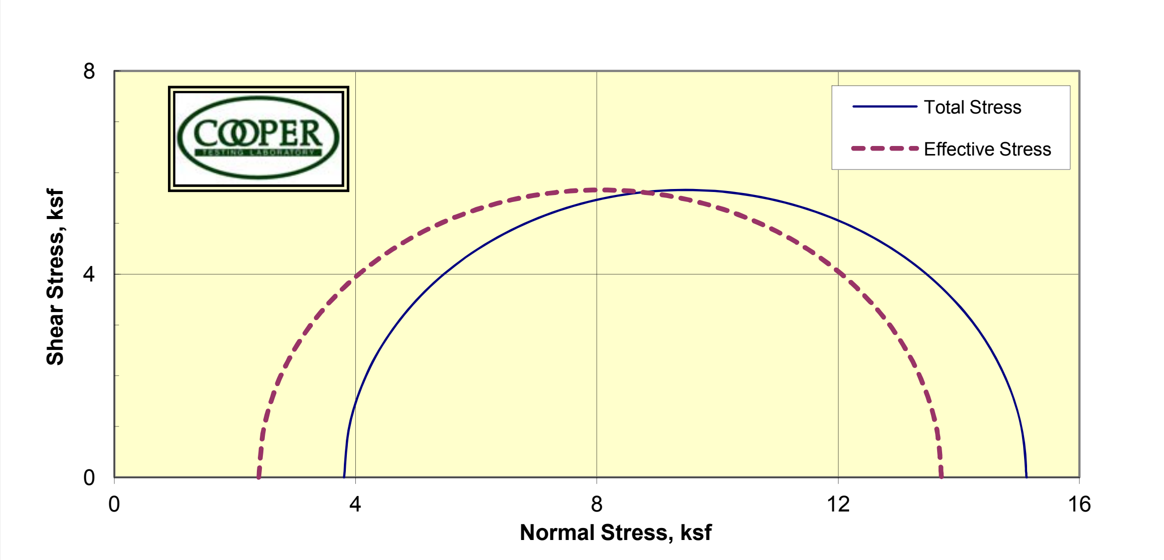
Project: Felkay - G0058

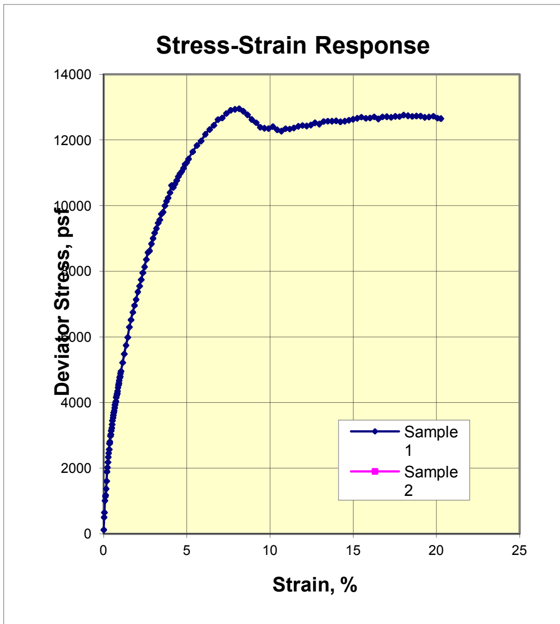
Sample 1)	B-4;3 @ 9-9.5'	Brown Mottled White SILT
Sample 2)	B-4;4 @ 16-16.5'	Pale Brown SILT Interming w/ Black Gravel
Sample 3)	B-4;6 @ 21-21.5'	Brown Mottled White SILT w/ Sand & Gravel

The pore pressure responded differently by going negative for sample #2. This may indicate that the sample was different from the others in possibly soil type, structure and moisture content.

Total C	**#DIV/0!**	**ksf**
Total phi	**#DIV/0!**	**degrees**
Eff. C	**#DIV/0!**	**ksf**
Eff. Phi	**#DIV/0!**	**degrees**

Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767





Sample:	1	2	3	4
MC, %	40.2			
DD, pcf	79.5			
Sat. %	97.0			
Void Ratio	1.118			
Diameter in	2.39			
Height, in	5.07			
	Final			
MC, %	39.4			
DD, pcf	81.6			
Sat. %	100.0			
Void Ratio	1.064			
Diameter, in	2.37			
Height, in	5.01			
Cell, psi	87.8			
BP, psi	61.4			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	11.318			
Excess PP	1.412			
Sigma 1	13.713			
Sigma 3	2.394			
P, ksf	8.054			
Q, ksf	5.659			
Stress Ratio	5.727			
Rate in/min	0.0005			

Total C	#DIV/0!	ksf
Total phi	#DIV/0!	degrees
Eff. C	#DIV/0!	ksf
Eff. Phi	#DIV/0!	degrees

Job No.: 026-498 **Date:** 9/23/2011

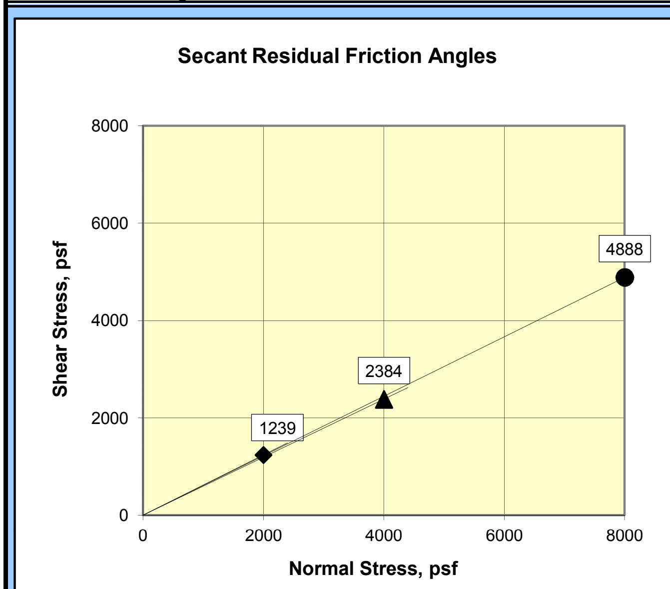
Client: Cotton, Shires & Associate: BY:DC

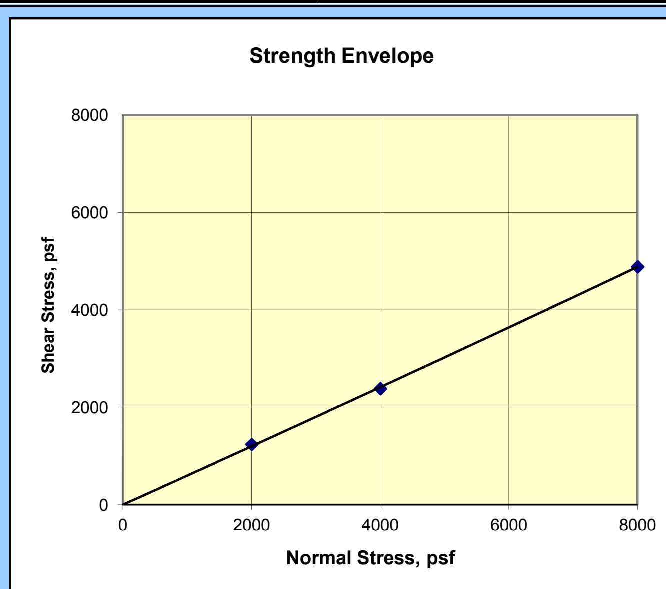
Project: Felkay - G0058

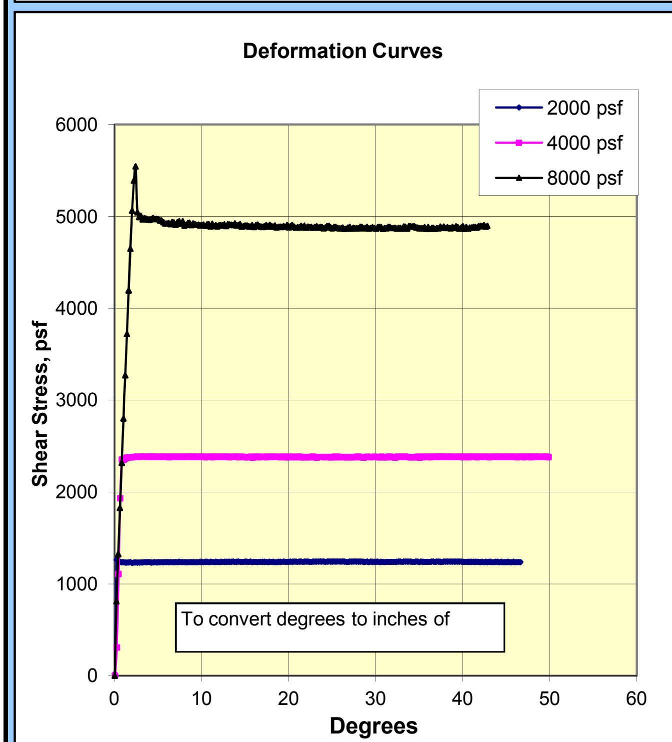
Sample 1)	B-2;MC-5 @ 26-26.5'	Pale Brown Sandy CLAY (Silty)
Sample 2)		
Sample 3)		

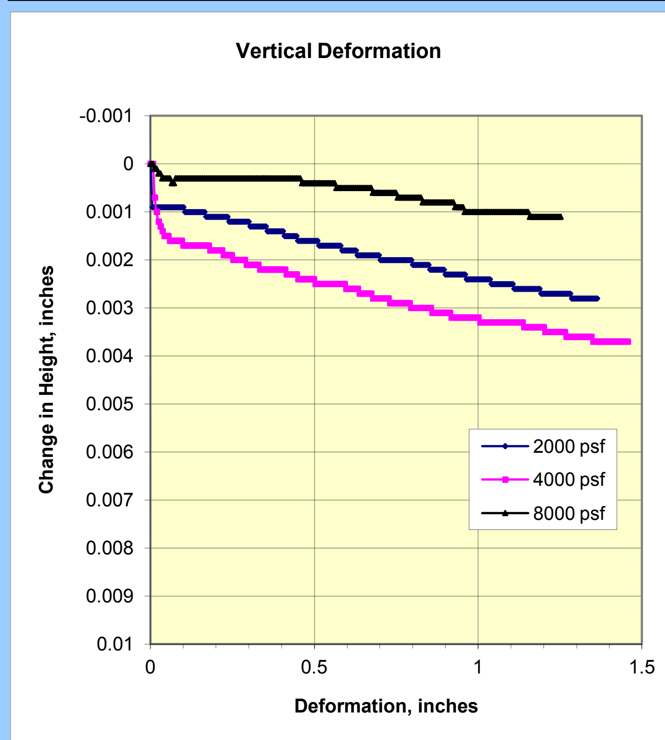
Drained, Residual Torsional Ring Shear Test ASTM D 6467

Job No.:	026-498	**Boring:**	B-5	**Date:**	9/23/2011	**Undisturbed:**	
Client:	Cotton Shires & Associates	**Sample:**	MC-2 & MC-3	**By:**	PJ	**Peak:**	
Project:	Felkay - G0058	**Depth:**	10.5-11.5'	**Checked:**	DC	**Residual:**	
Soil Type:	Brown Sandy SILT			**Clay, %:**	14	**Fully Softened:**	X
Remarks:	Sample prepared over the #200 sieve. A small friction correction was applied to each point			**LL:**	17	**Peak:**	
Normal Stress, psf	2000	4000	8000	**PL:**	14	**Residual:**	X
Secant Phi, deg.	32	31	31				









Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D 2937)

CTL Job No: 026-498 **Project No.** G0058 **By:** RU

Client: Cotton, Shires & Assoc. **Date:** 09/20/11

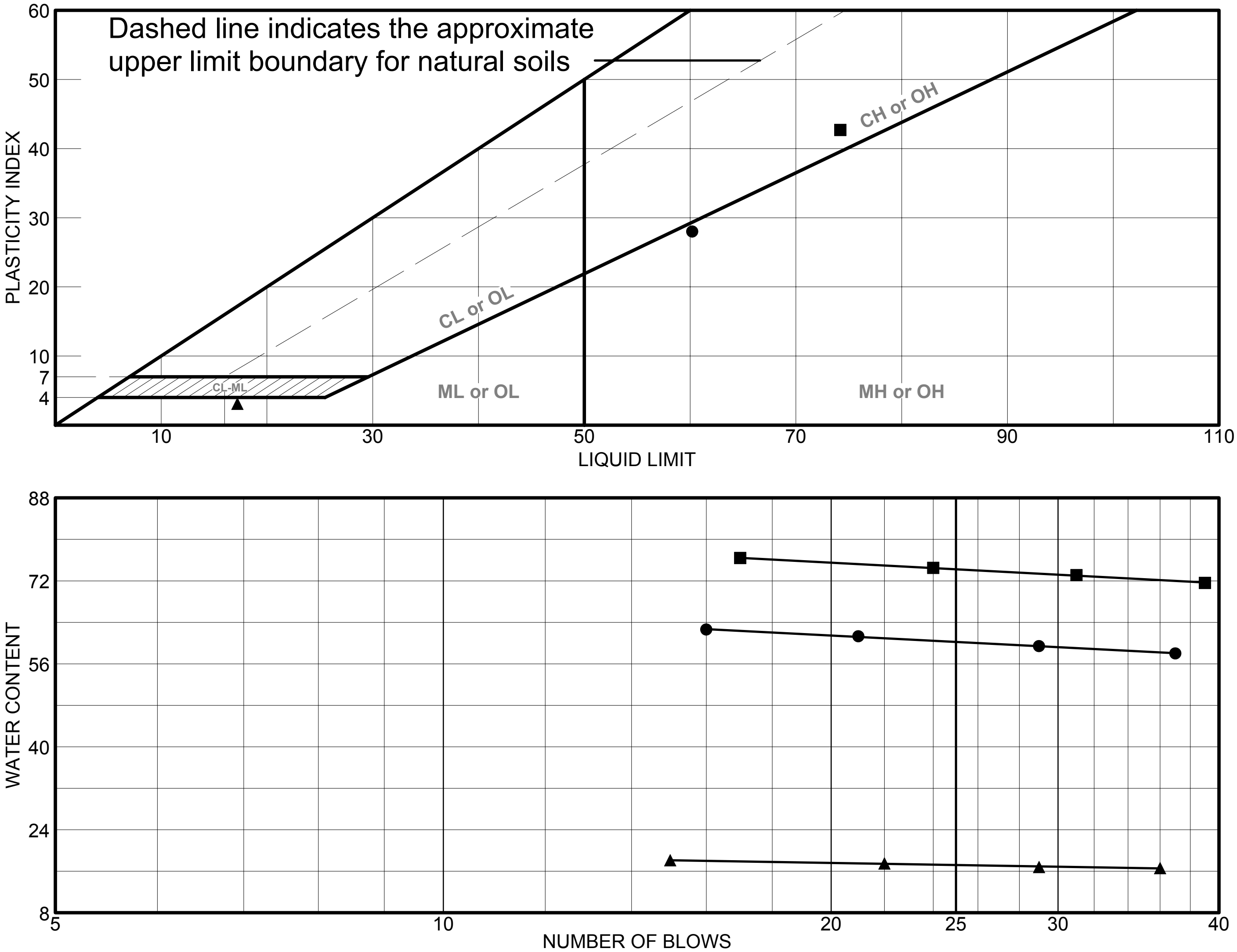
Project Name: Felkay **Remarks:**

Boring:	B-1	B-1	B-5					
Sample:	MC-7	MC-13	MC-1					
Depth, ft:	11-11.5	26-26.5	6-6.5					
Visual Description:	Very Dark Brown Clayey SAND w/ Gravel (Weathered Claystone)	Olive Brown CLAY	Dark Brown Clayey SAND (Silty)					
Actual G_s								
Assumed G_s	2.70	2.70	2.70					
Moisture, %	49.3	43.4	9.9					
Wet Unit wt, pcf	88.9	108.4	114.5					
Dry Unit wt, pcf	59.6	75.6	104.2					
Dry Bulk Dens.pb, (g/cc)	0.95	1.21	1.67					
Saturation, %	72.7	95.2	43.0					
Total Porosity, %	64.7	55.2	38.2					
Volumetric Water Cont, Θw	47.0	52.5	16.4					
Volumetric Air Cont., Θa	17.7	2.7	21.8					
Void Ratio	1.83	1.23	0.62					
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive Brown Sandy Elastic SILT, trace Gravel	60.2	32.2	28.0			
■	Olive Brown Fat CLAY	74.2	31.5	42.7			
▲	Brown Sandy SILT	17.2	14.1	3.1	95.5	52.6	ML

Project No. 026-498 **Client:** Cotton, Shires & Associates

Project: Felkay - G0058

● **Source:** B-1 **Sample No.:** MC-11 **Elev./Depth:** 21.0-21.5'

■ **Source:** B-1 **Sample No.:** MC-14 **Elev./Depth:** 25.5-26.0'

▲ **Source:** B-5 **Sample No.:** MC-2+MC-3 **Elev./Depth:** 10.5-11.5'

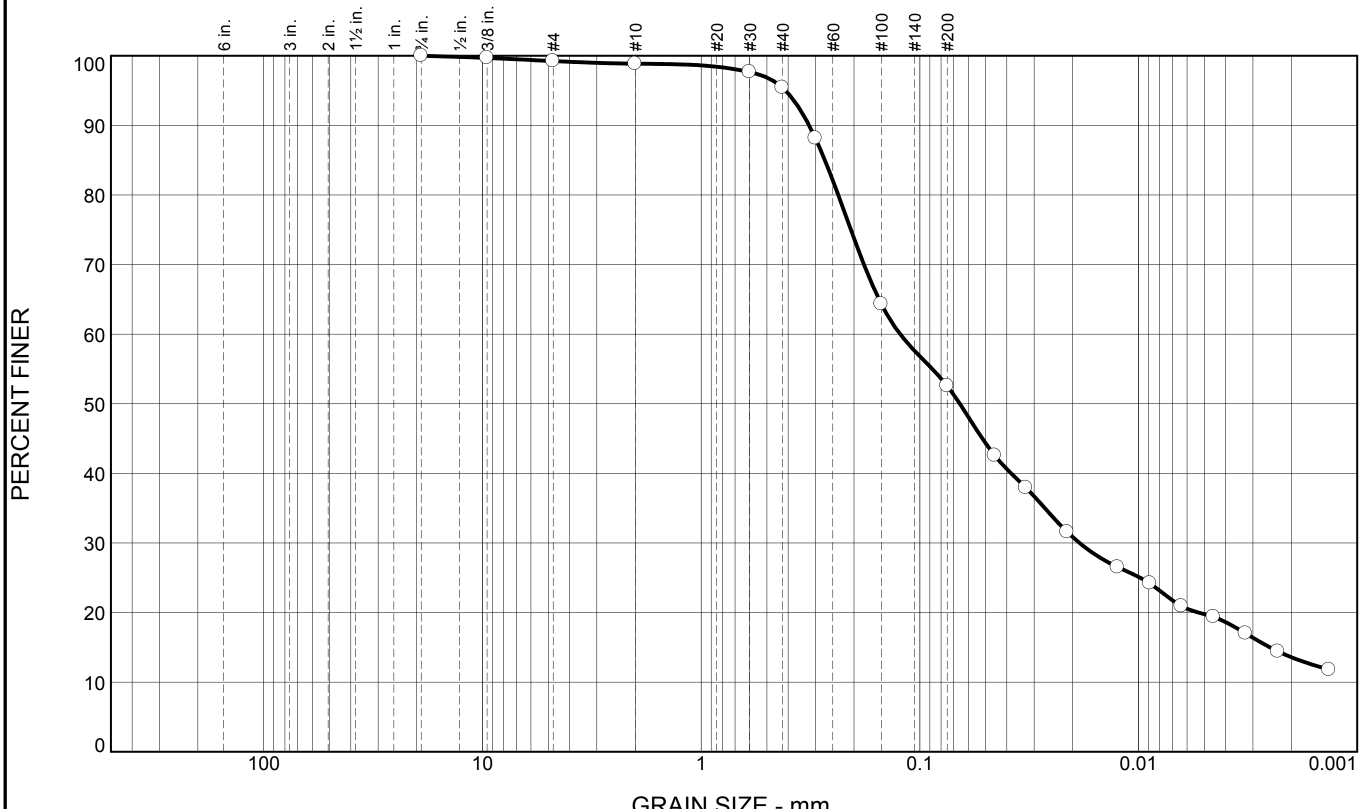
Remarks:
●
■
▲

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.8	0.3	3.5	42.8	39.0	13.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	99.7		
#4	99.2		
#10	98.9		
#30	97.7		
#40	95.4		
#50	88.1		
#100	64.3		
#200	52.6		
0.0455 mm.	42.6		
0.0327 mm.	37.9		
0.0212 mm.	31.6		
0.0125 mm.	26.5		
0.0089 mm.	24.2		
0.0064 mm.	20.9		
0.0045 mm.	19.4		
0.0032 mm.	17.0		
0.0023 mm.	14.4		
0.0013 mm.	11.8		

* (no specification provided)

Soil Description

Brown Sandy SILT

Atterberg Limits

PL= 14 LL= 17 PI= 3

Coefficients

D_{90}= 0.3207 D_{85}= 0.2717 D_{60}= 0.1230
D_{50}= 0.0656 D_{30}= 0.0186 D_{15}= 0.0025
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Source of Sample: B-5 **Depth:** 10.5-11.5'
Sample Number: MC-2+MC-3

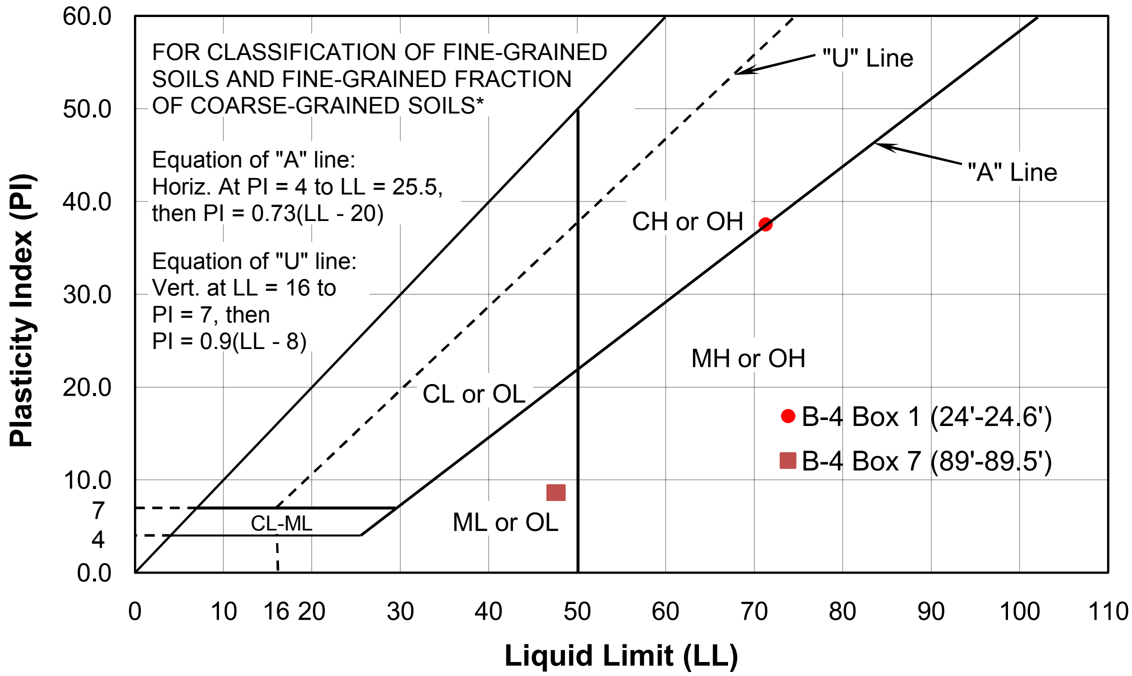
Date: 9/16/11

COOPER TESTING LABORATORY

Client: Cotton, Shires & Associates
Project: Felkay - G0058

Project No: 026-498 **Figure**

SUMMARY OF ATTERBERG LIMITS



SAMPLE DESCRIPTION	BORING No./ SAMPLE No.	DEPTH, Ft.	LIQUID LIMIT, %	PLASTICITY INDEX, %	USCS SYMBOL
Claystone; greyish brown with orange	B-4/Box 1	24'-24.6'	71.3	37.5	CH
Sandy Silty Claystone; medium yellow brown	B-4/Box 7	89'-89.5'	47.6	8.5	ML

*Reference: 1995 Annual Book of ASTM Standards; ASTM Designation D4318: Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

COTTON, SHIRES, & ASSOCIATES, INC.

Atterberg Limits

Project: Felkay Proj No.: G0058 Location: 1921 ECDLL; Above Headscarp

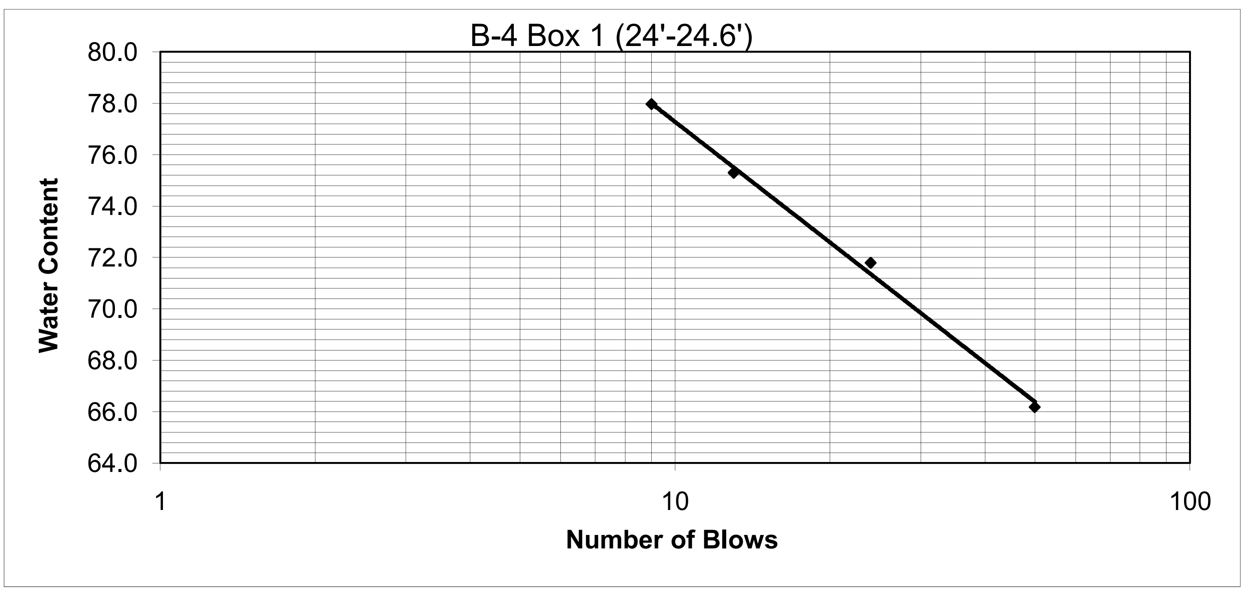
Hole: B-4
Sample: B-4 Box 1 (24'-24.6') Description of Sample: Claystone; greyish brown with orange

Tested by: JN Date of Testing: 8/22/2011

	LIQUID LIMIT			
Trial Number	1	2	3	4
Tin Number	42	36	L-3	15
Weight of Tin	1.6	1.63	1.68	1.57
Number of Blows	9	13	24	50
Tin + Wet Soil	21.23	20.72	26.04	22.01
Tin+ Dry Soil	12.63	12.52	15.86	13.87
Weight Water	8.6	8.2	10.18	8.14
Weight of Dry Soil	11.03	10.89	14.18	12.3
Moisture Content (%)	78.0	75.3	71.8	66.2

	PLASTIC LIMIT			
Trial Number	1	2	3	4
Tin Number	58	51	8	n/a
Weight of Tin	1.61	1.65	1.62	
Tin + Wet Soil	3.6	3.8	4.94	
Tin+ Dry Soil	3.09	3.28	4.08	
Weight Water	0.51	0.52	0.86	
Weight of Dry Soil	1.48	1.63	2.46	
Moisture Content (%)	34.5	31.9	35.0	

LL 71.3
PL 33.8
PI 37.5



Unified Soil Classification: CH

COTTON, SHIRES, & ASSOCIATES, INC.

Atterberg Limits

Project Felkay Proj No. G0058 Location 1921 ECDLL; Above Headscarp

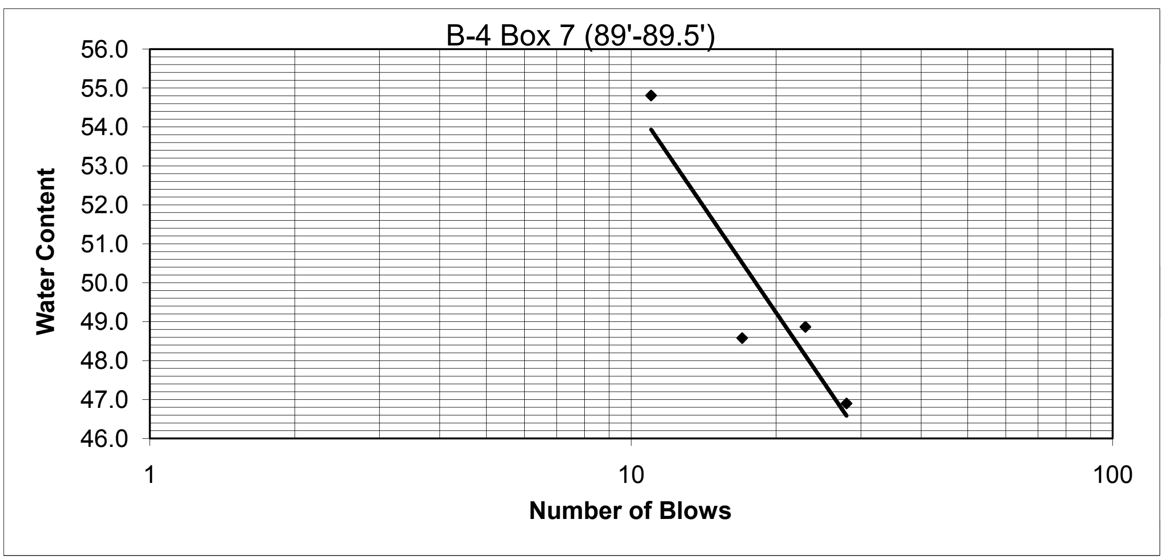
Hole B-4
Sample B-4 Box 7 (89'-89.5') Description of Sample Sandy Silty Claystone; medium yellow brown

Tested by JN Date of Testing 8/22/2011

	LIQUID LIMIT			
Trial Number	1	2	3	4
Tin Number	RA	18	12	13
Weight of Tin	1.6	1.54	1.6	1.57
Number of Blows	11	17	23	28
Tin + Wet Soil	14.96	25.61	28.53	20.3
Tin+ Dry Soil	10.23	17.74	19.69	14.32
Weight Water	4.73	7.87	8.84	5.98
Weight of Dry Soil	8.63	16.2	18.09	12.75
Moisture Content (%)	54.8	48.6	48.9	46.9

	PLASTIC LIMIT			
Trial Number	1	2	3	4
Tin Number	H-5	8	0	n/a
Weight of Tin	1.55	1.51	1.58	
Tin + Wet Soil	5.07	3.65	3.55	
Tin+ Dry Soil	4.04	3.07	3	
Weight Water	1.03	0.58	0.55	
Weight of Dry Soil	2.49	1.56	1.42	
Moisture Content (%)	41.4	37.2	38.7	

LL 47.6
PL 39.1
PI 8.5



Unified Soil Classification: ML

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

APPENDIX C

INSTRUMENTATION

Summary of Piezometer Data
Summary of Inclinometer Data

APPENDIX C

MONITORING

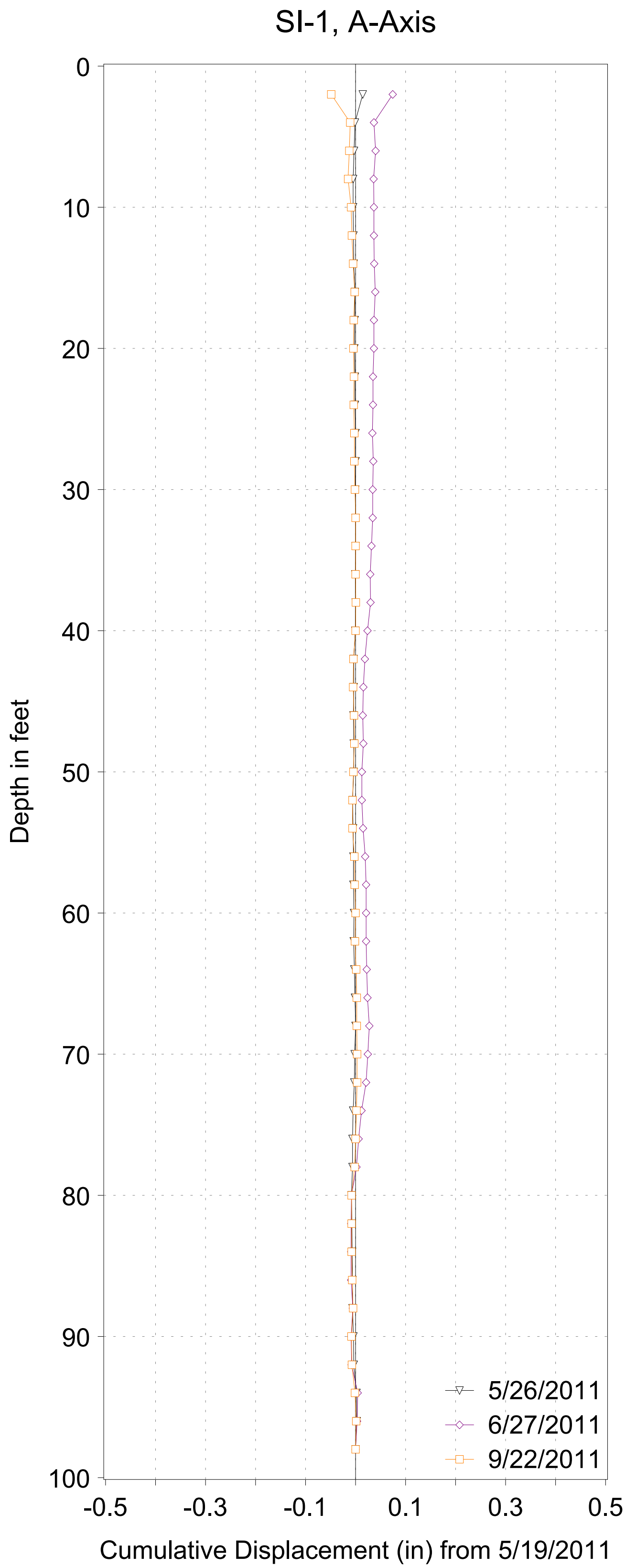
Inclinometers

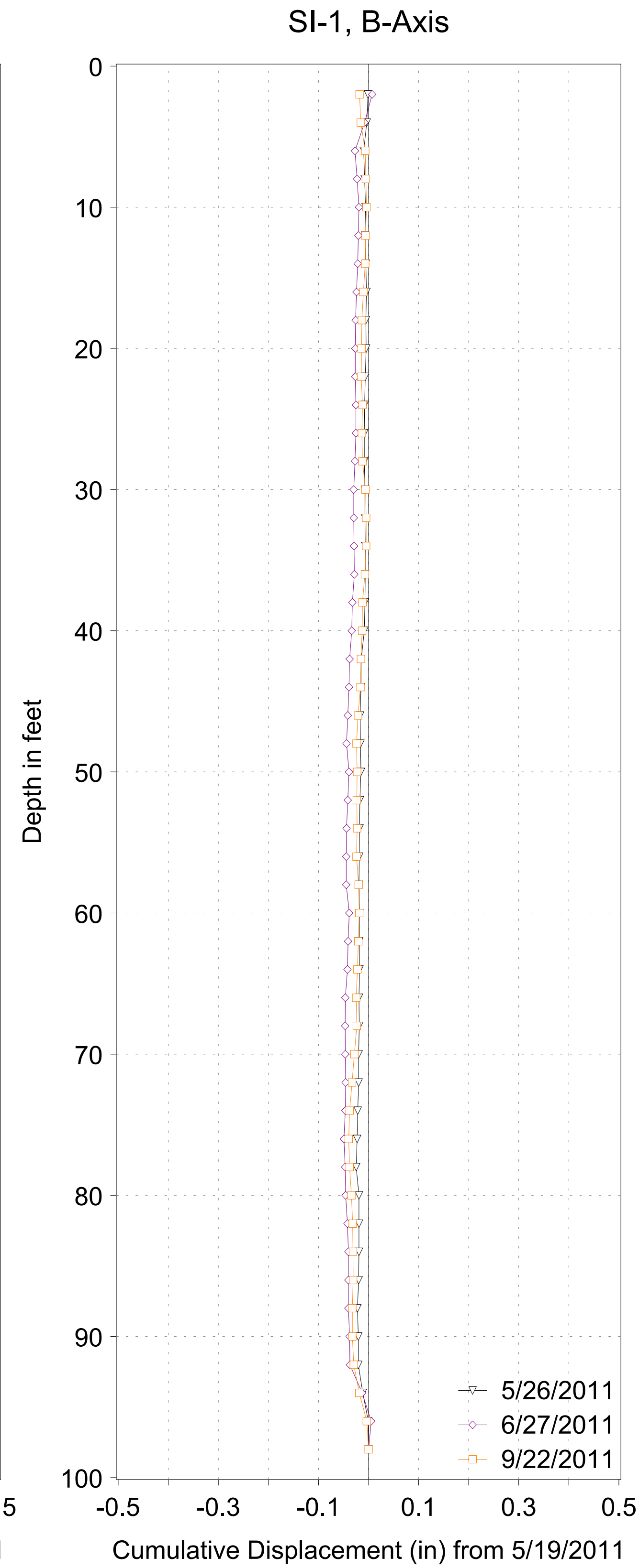
CSA installed three inclinometers on the parcel, identified as SI-1, SI-2, and SI-3, which have functioned throughout the measurement period and are proposed to remain in place and be periodically monitored through the project regulatory review process. Inclinometer SI-1 is located upslope of the proposed residential envelope and upslope of the 1978 landslide, east of the parcel driveway near elevation 131 feet. SI-2 is located within the proposed residential development envelope and within the headscarp of the 1978 landslide, near elevation 120 feet. SI-3 is located near the downslope edge of the proposed residential development envelope and within the upper portion of the 1978 landslide, near elevation 88 feet. The three inclinometers, with 2.75-inch casing diameter, were installed to near the full depth of small diameter boreholes B-1, B-2, and B-3, to depths of 98 feet in SI-1, 94 feet in SI-2, and 100 feet in SI-3, and were grouted in-place. The inclinometers were installed between May 11 and May 19, 2011, with initial readings taken between May 19 and June 1, 2011. The most recent reading was performed on January 5, 2012. The inclinometer data indicate that there has been no movement, within the statistical accuracy limits of the instruments, either of the hillside above the 1978 landslide or of the 1978 landslide area near SI-3.

Piezometers

CSA installed three piezometers in small-diameter borings B-1, B-2, and B-3, each consisting 2 vibrating wire piezometer sensors that are used to record in-situ groundwater pore pressures. The piezometers have functioned throughout the measurement period and are proposed to remain in place and be periodically monitored through the project regulatory review process. Two of the piezometers (SI-1 and SI-2) are located upslope of the 1978 landslide debris, and 1 of these piezometers (SI-3) is located with the 1978 landslide debris. Two vibrating wire piezometer sensors were installed in each of the three boreholes, with one at a mid-range depth of between 38 and

45 feet, and one deep sensor between 88 and 93 feet below ground surface. Piezometers were attached to the sides of the inclinometer casing during installation and grouted in place. Water levels were recorded in these instruments (see attached piezometer graphs in this Appendix) between mid-May 2011 and early January 2012. Groundwater levels have experienced little fluctuation during the monitoring period, and generally correspond with the transition from oxidized to unoxidized bedrock between 22 and 35 feet below the ground surface. The deep and shallow sensors reveal groundwater levels nearly at the same respective elevations, indicating that neither artesian conditions nor perched water tables were present at the site during the measurement period.

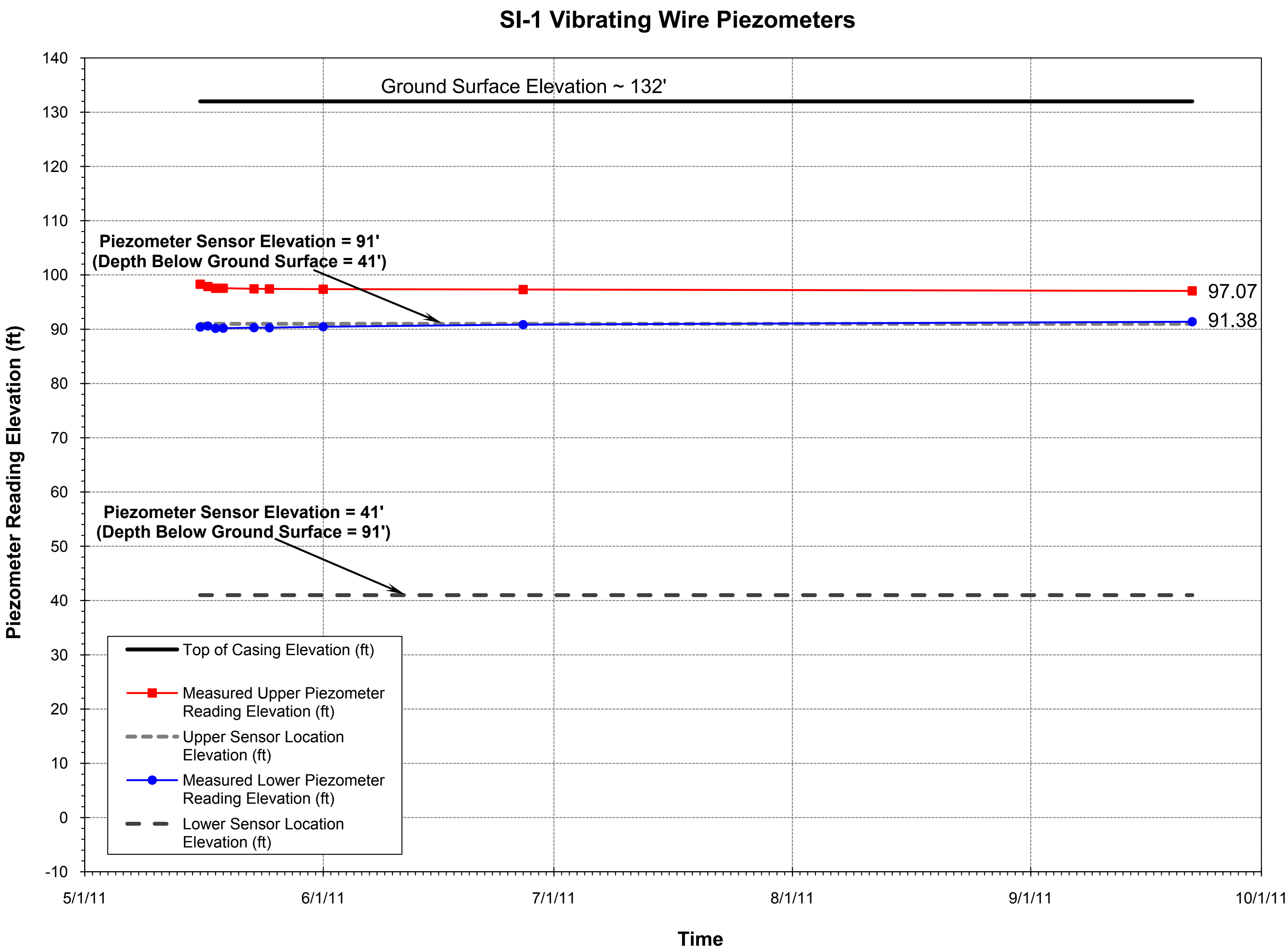


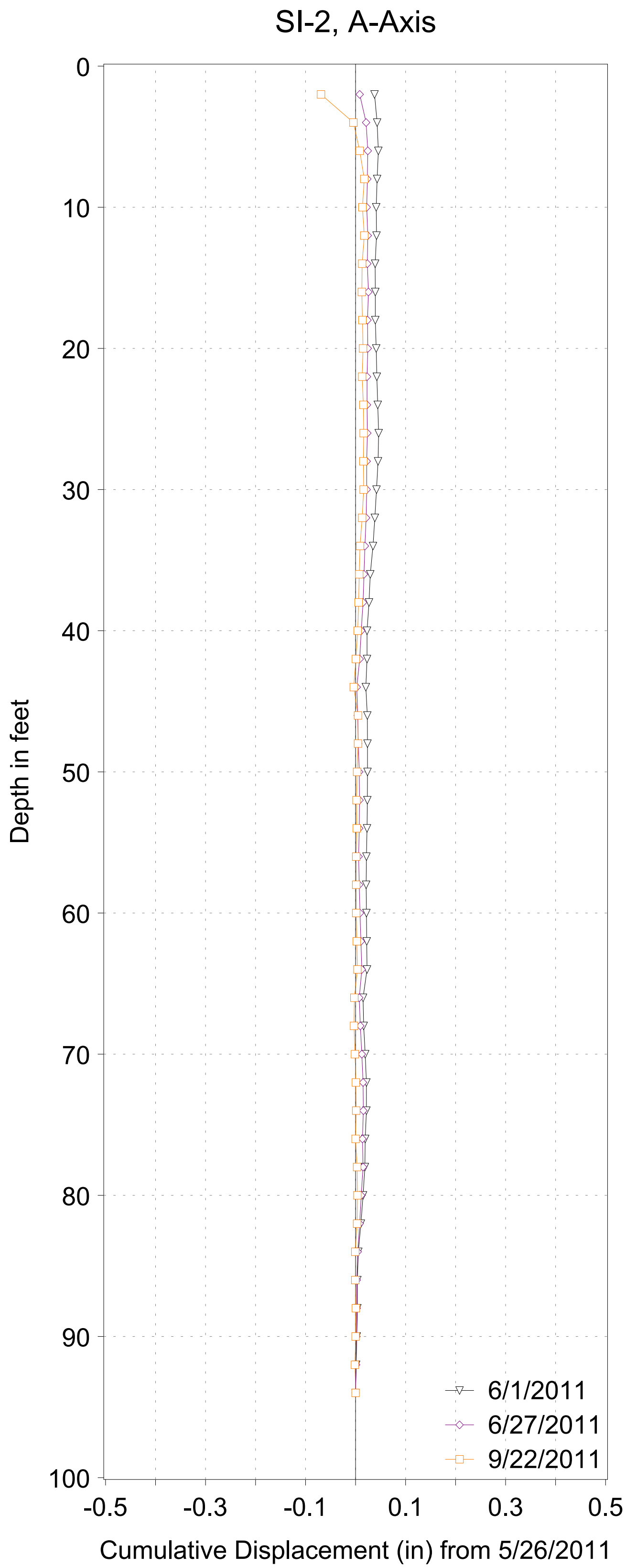


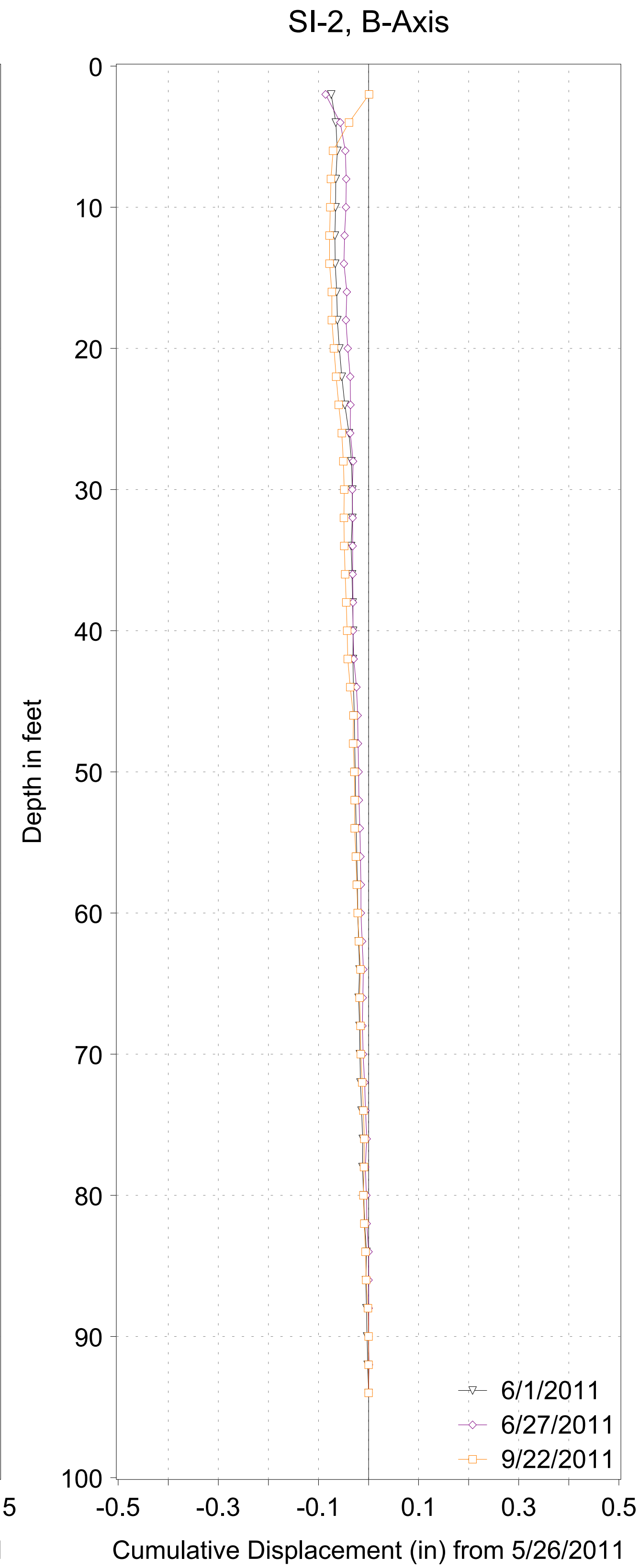
1925 El Camino De La Luz
A+ Direction: S10E
Top of Casing Elevation=132'

CSA/SI-1

Cotton, Shires and Associates, Inc.
Consulting Engineers and Geologists


SI-1 Vibrating Wire Piezometers
Ground Surface Elevation ~ 132'
Piezometer Sensor Elevation = 91'
(Depth Below Ground Surface = 41')
97.07
91.38
Piezometer Sensor Elevation = 41'
(Depth Below Ground Surface = 91')
Top of Casing Elevation (ft)
Measured Upper Piezometer Reading Elevation (ft)
Upper Sensor Location Elevation (ft)
Measured Lower Piezometer Reading Elevation (ft)
Lower Sensor Location Elevation (ft)
Piezometer Reading Elevation (ft)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-10
5/1/11
6/1/11
7/1/11
8/1/11
9/1/11
10/1/11
Time





1925 El Camino De La Luz
A+ Direction: S48W
Top of Casing Elevation=119'

CSA/SI-2

COTTON, SHIRES AND ASSOCIATES, INC.
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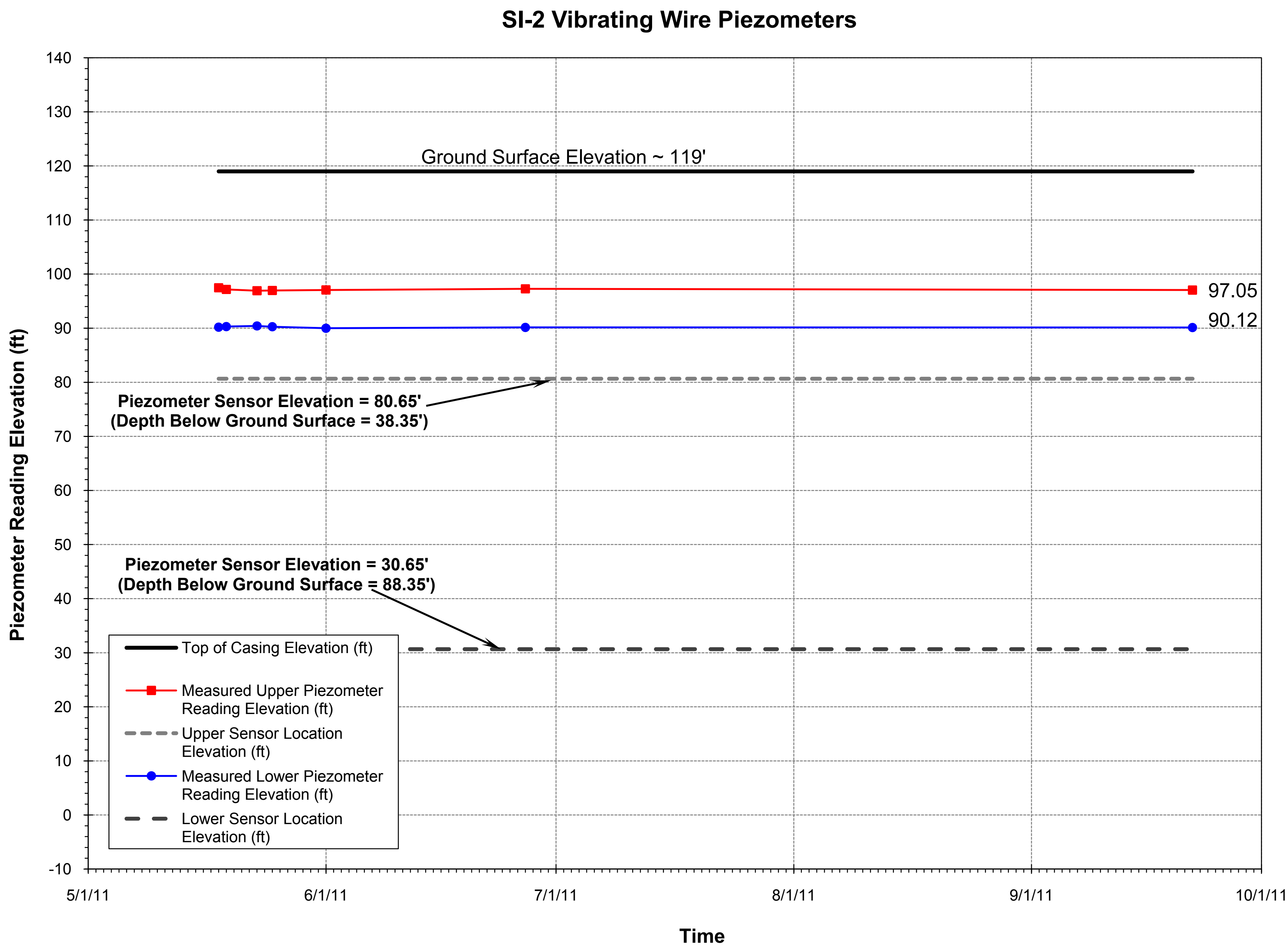
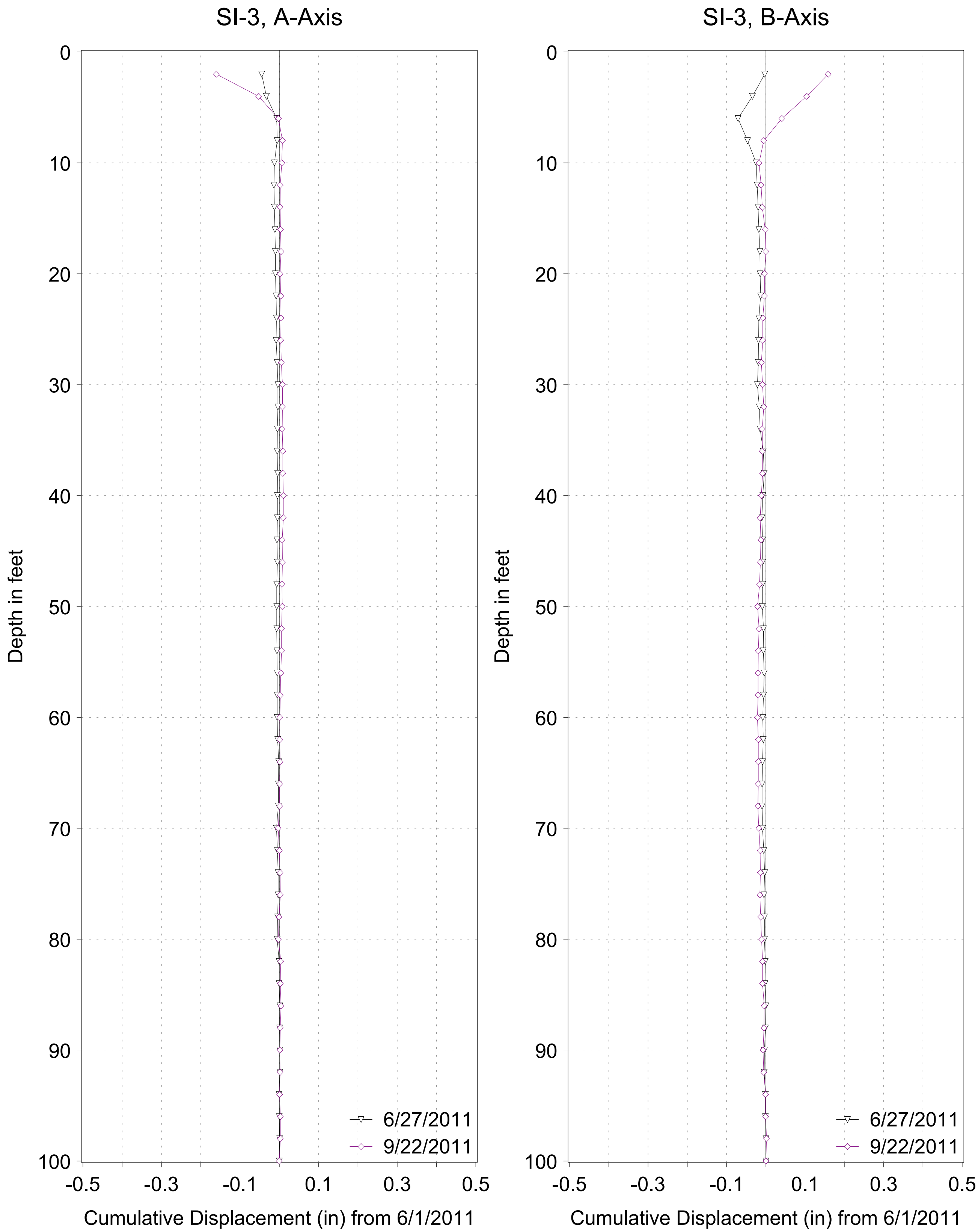

SI-2 Vibrating Wire Piezometers
Ground Surface Elevation ~ 119'
97.05
90.12
Piezometer Sensor Elevation = 80.65'
(Depth Below Ground Surface = 38.35')
Piezometer Sensor Elevation = 30.65'
(Depth Below Ground Surface = 88.35')
Top of Casing Elevation (ft)
Measured Upper Piezometer Reading Elevation (ft)
Upper Sensor Location Elevation (ft)
Measured Lower Piezometer Reading Elevation (ft)
Lower Sensor Location Elevation (ft)
Piezometer Reading Elevation (ft)
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-10
5/1/11
6/1/11
7/1/11
8/1/11
9/1/11
10/1/11
Time



1925 El Camino De La Luz
A+ Direction: S38W
Top of Casing Elevation=87'

CSA/SI-3

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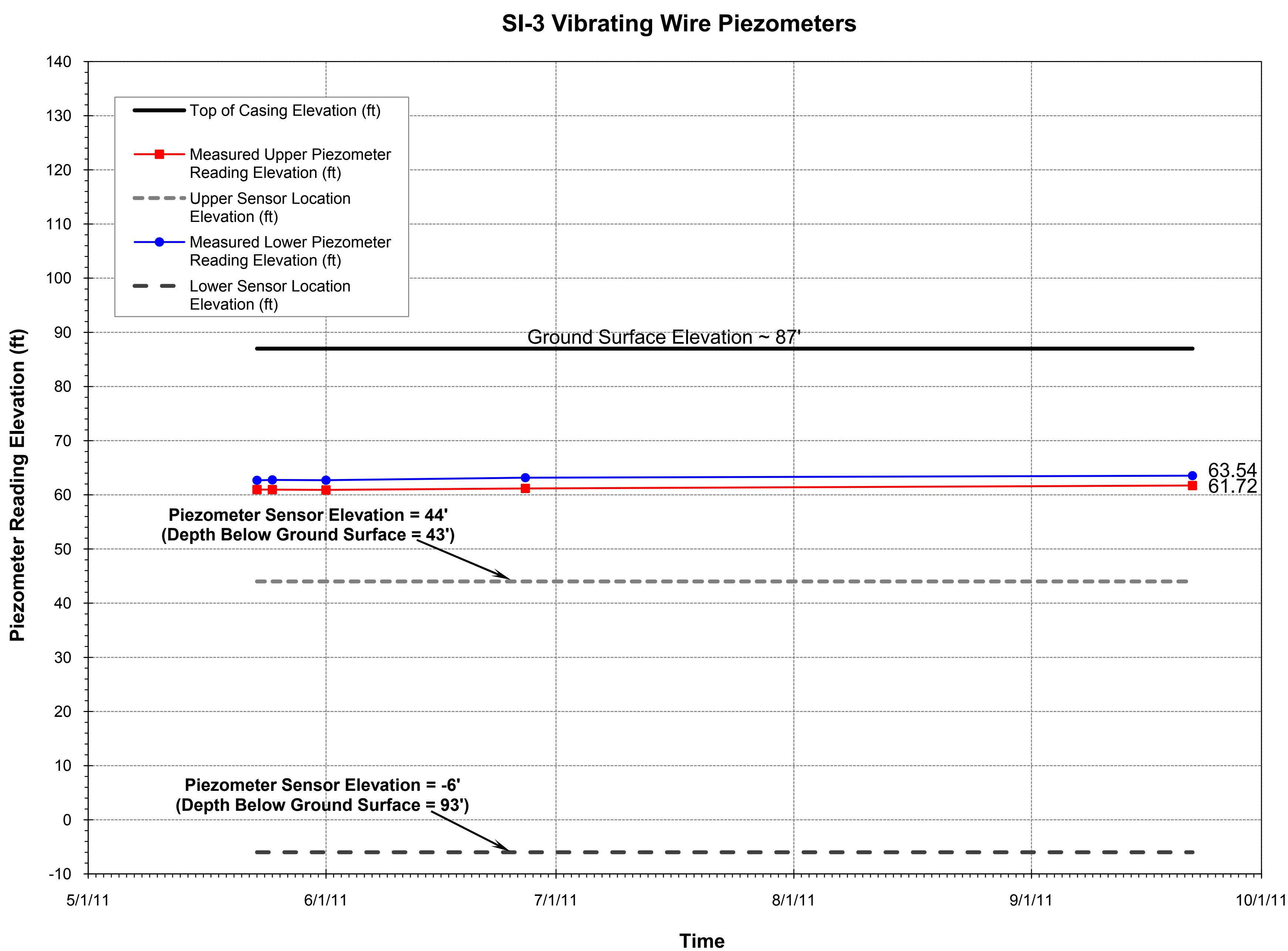
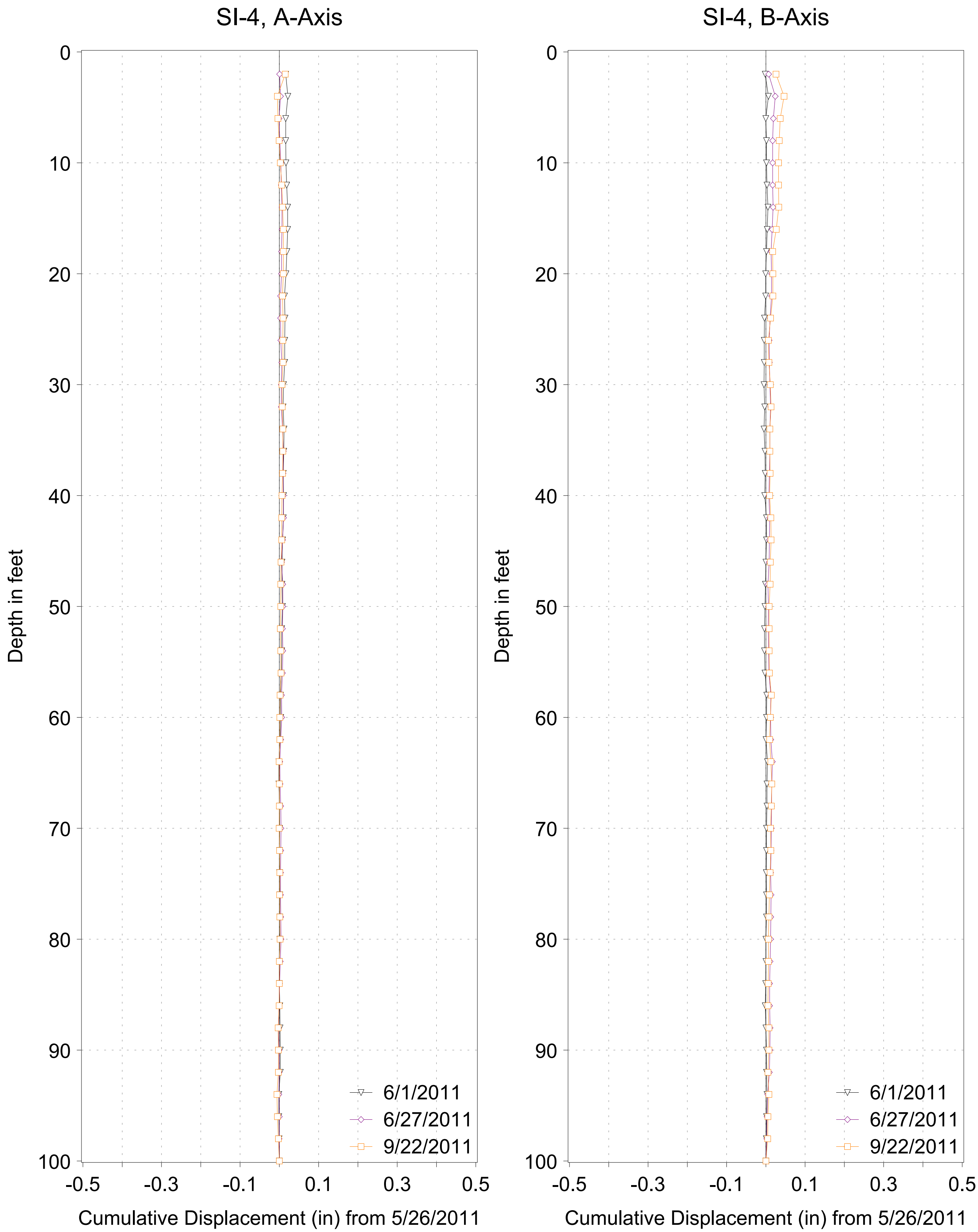


Cotton, Shires and Associates, Inc.
Consulting Engineers and Geologists



1921 El Camino De La Luz
A+ Direction: S64W
Top of Casing Elevation=123'

CSA/SI-4

COTTON, SHIRES AND ASSOCIATES, INC.
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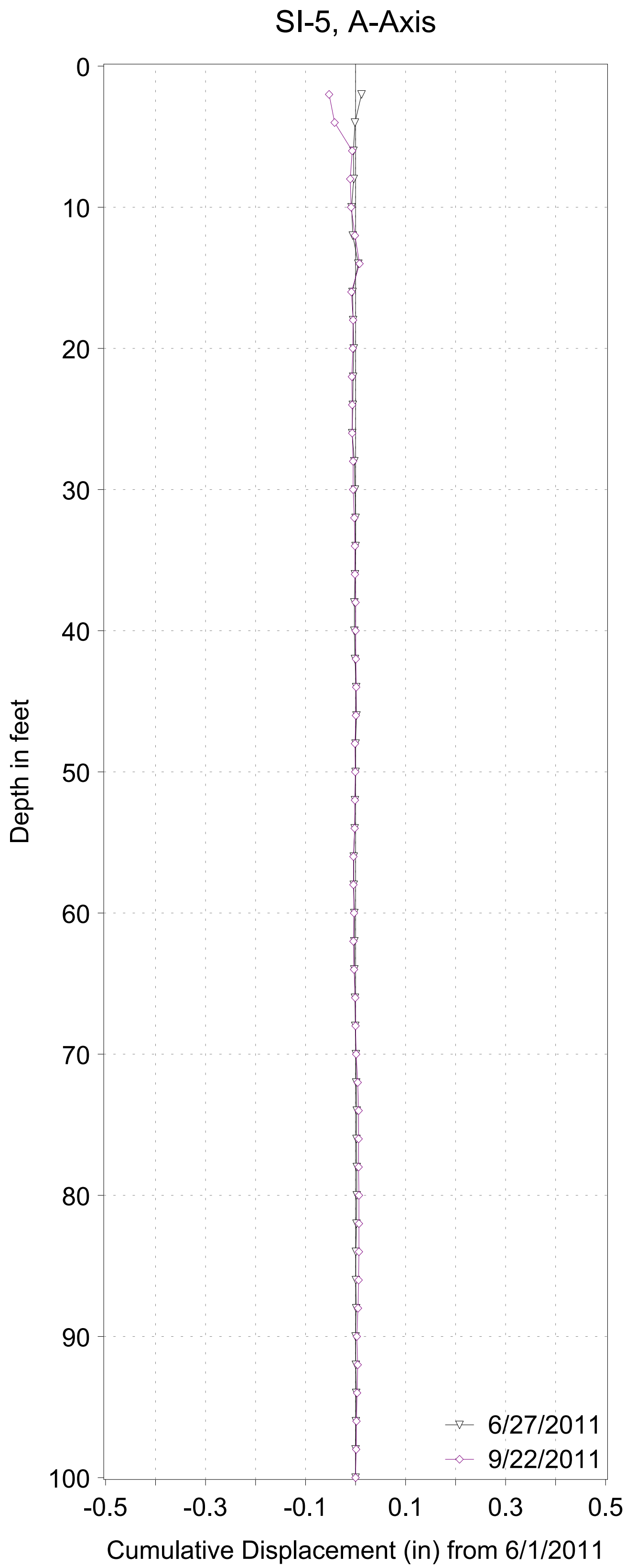

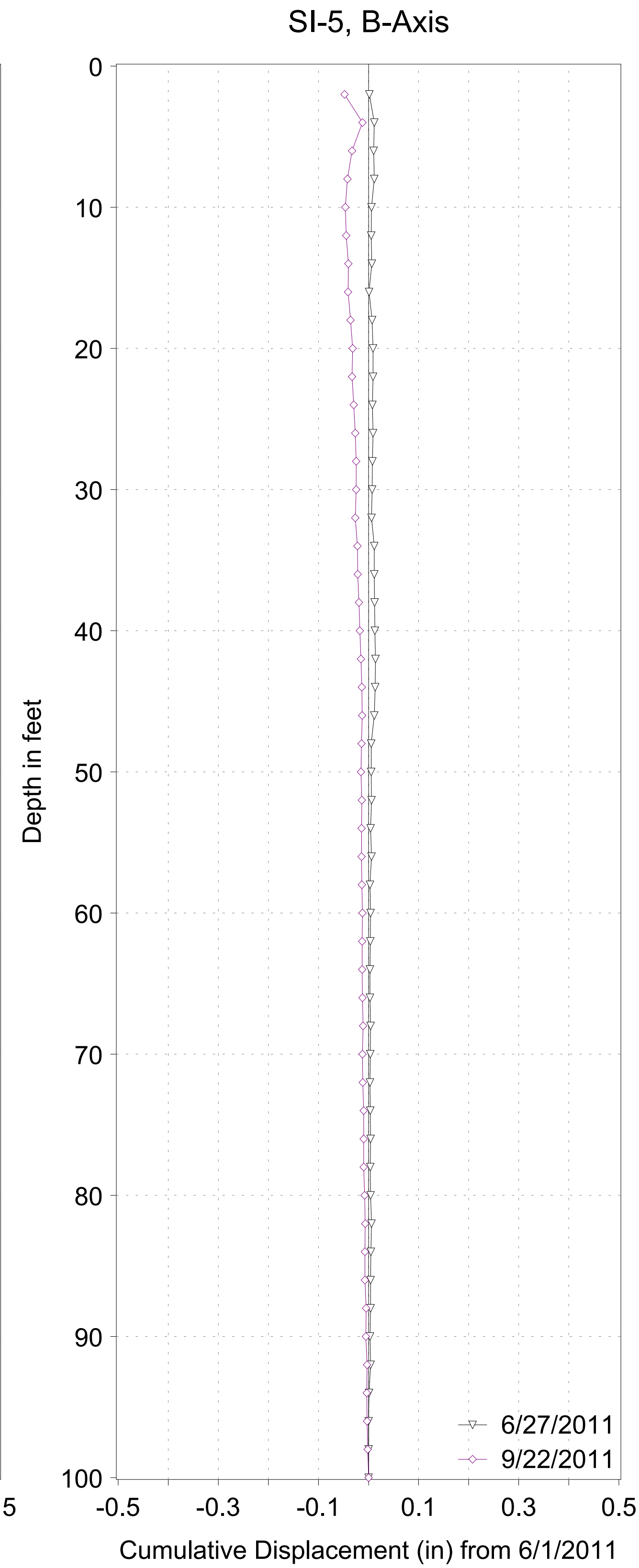

1921 El Camino De La Luz

A+ Direction: S06W

Top of Casing Elevation=91'

CSA/SI-5

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CONSULTING ENGINEERS AND GEOLOGISTS

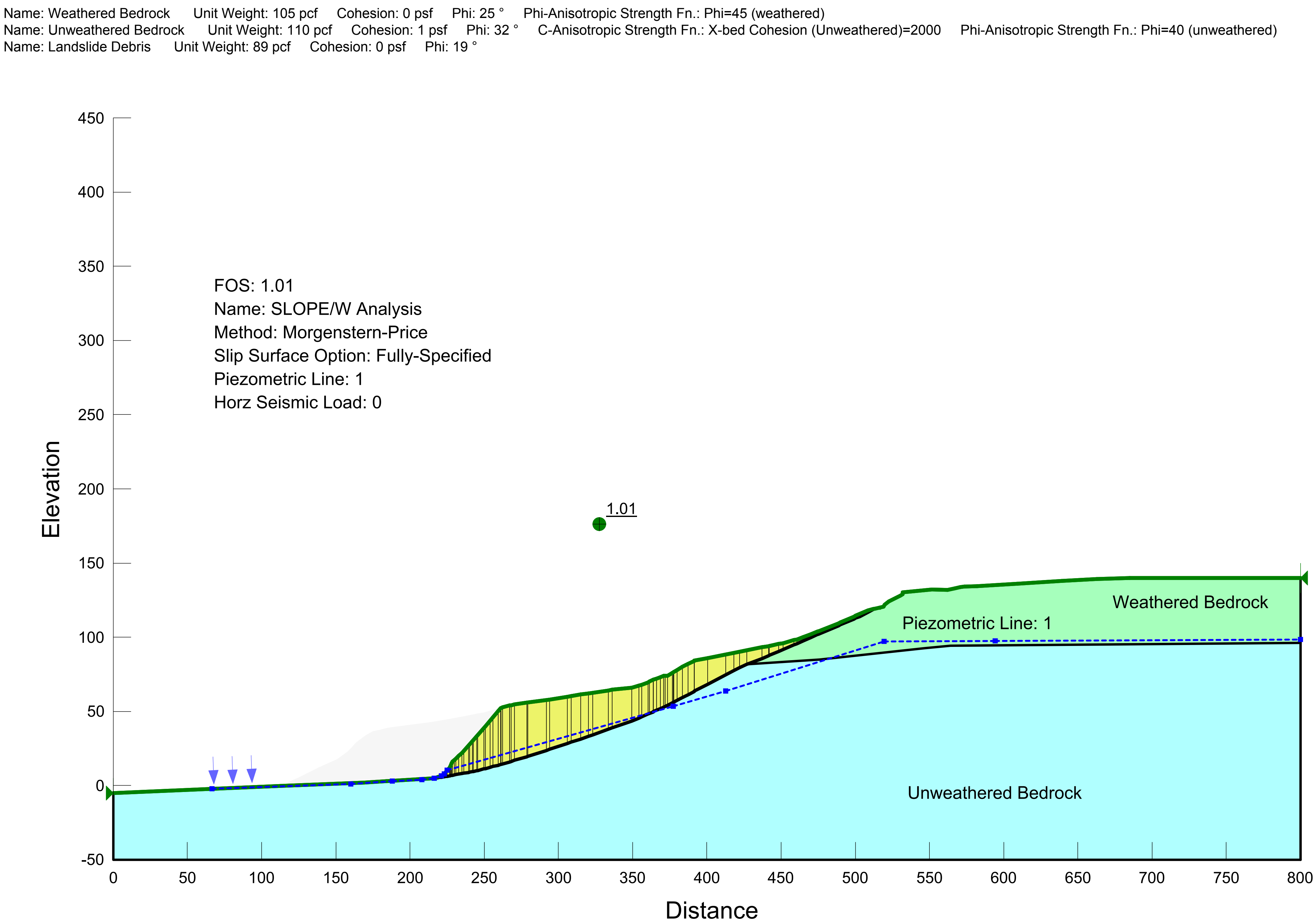
APPENDIX D

SLOPE STABILITY ANALYSIS OUTPUT FILES

APPENDIX D

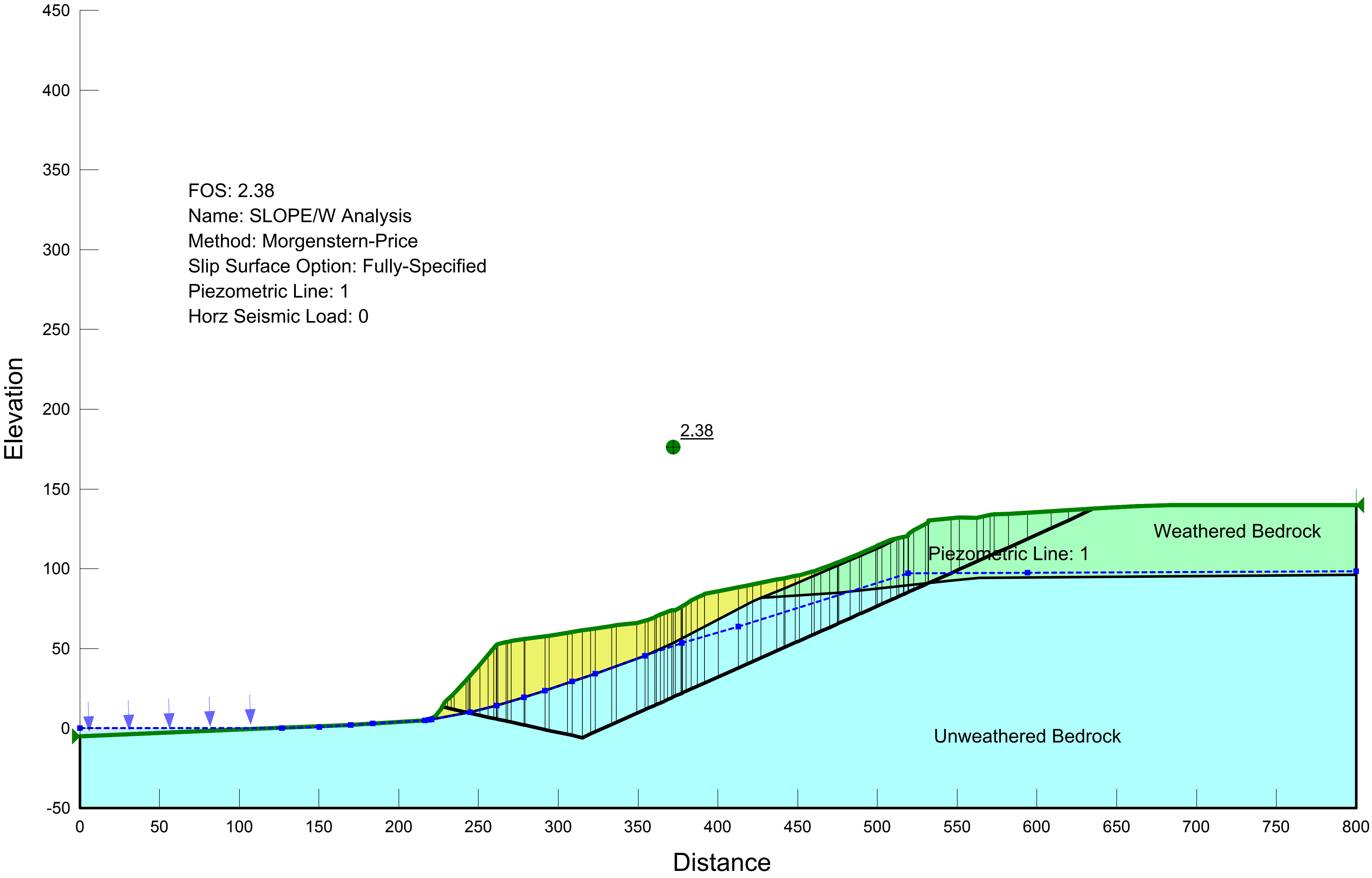
SLOPE STABILITY ANALYSIS OUTPUT FILES

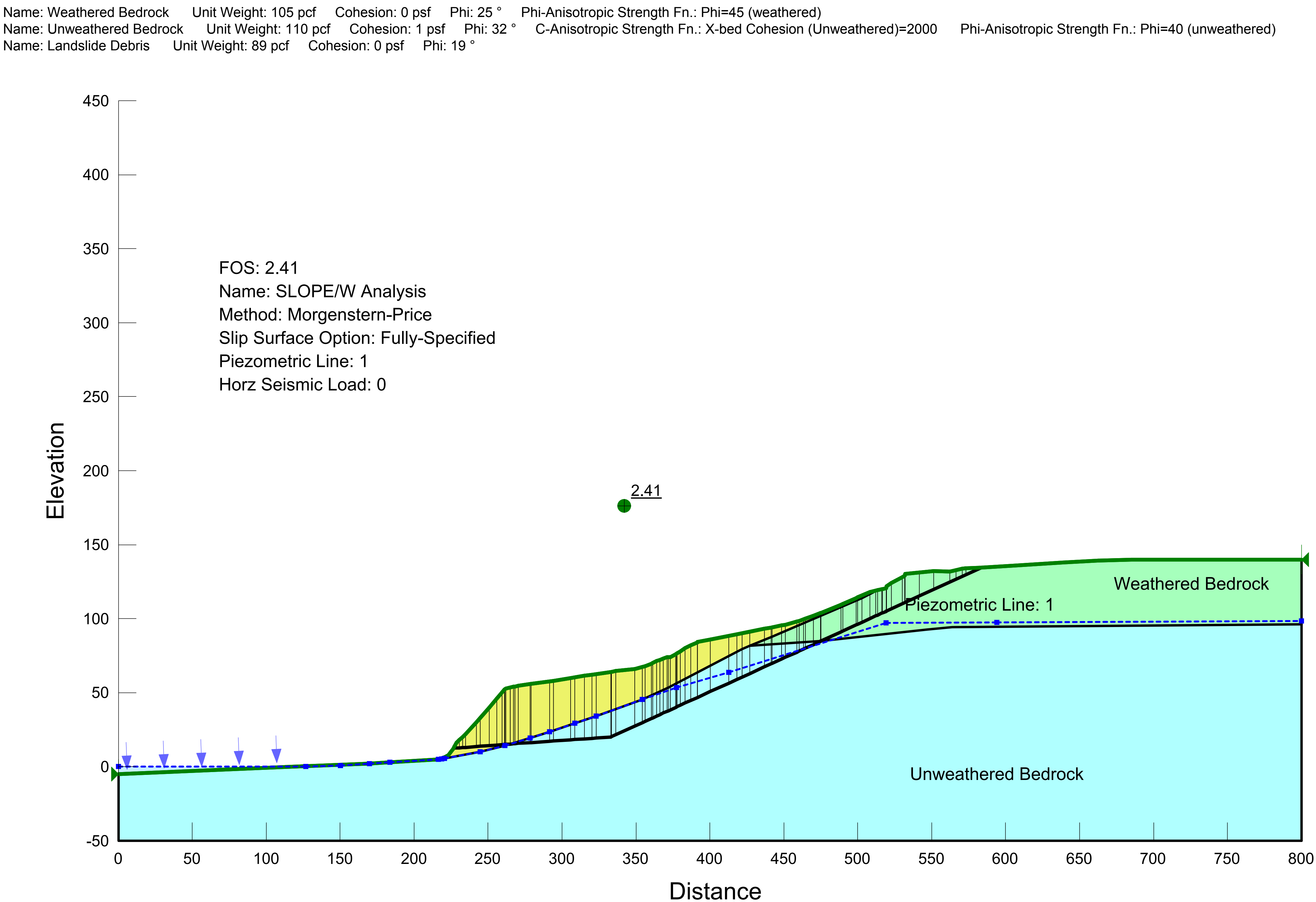
SSA Run #	Slope Condition Analyzed	Lower SP Wall (kips)	Upper SP Wall (kips)	TB (kips)	FS
1	Back-Calculation of existing slide debris surface	N/A	N/A	N/A	1.01
2	Slide debris In place (Critical Surface obtained from post-construction condition Run 5)	N/A	N/A	N/A	2.38
2A	Potential bedding failure with slide debris in place	N/A	N/A	N/A	2.41
3	Same as Run 2, but with slide debris removed (Critical Surface obtained from post-construction condition Run 5)	N/A	N/A	N/A	1.66
3A	Potential bedding failure with slide debris removed (same Critical Surface as Run 2A)	N/A	N/A	N/A	1.96
3B	Deep Block Search with debris removed	N/A	N/A	N/A	1.58
4	Shallow Surface from Run 7, no SP or slide debris	N/A	N/A	N/A	1.39
4A	Same as 4, but with slide debris in place	N/A	N/A	N/A	1.88
5	Critical Deep Surface, post construction of SP walls/ TBs / house loads, slide debris removed downslope of lower SP	40	50	100	1.68
6	Same Critical Surface and construction as Run 5, Seismic with k=0.15	40	50	100	1.23
7	Potential failure through upper shear pin wall (post construction, slide debris removed downslope of lower SP)	40	50	100	2.15
8	Same as Run 7, now with seismic k=0.15	40	50	100	1.24
9	Same as Run 5, with landslide debris in place downslope of lower SP	40	50	100	2.42
10	Same as Run 5, with landslide debris in place downslope of lower SP, now with seismic k=0.15	40	50	100	1.66



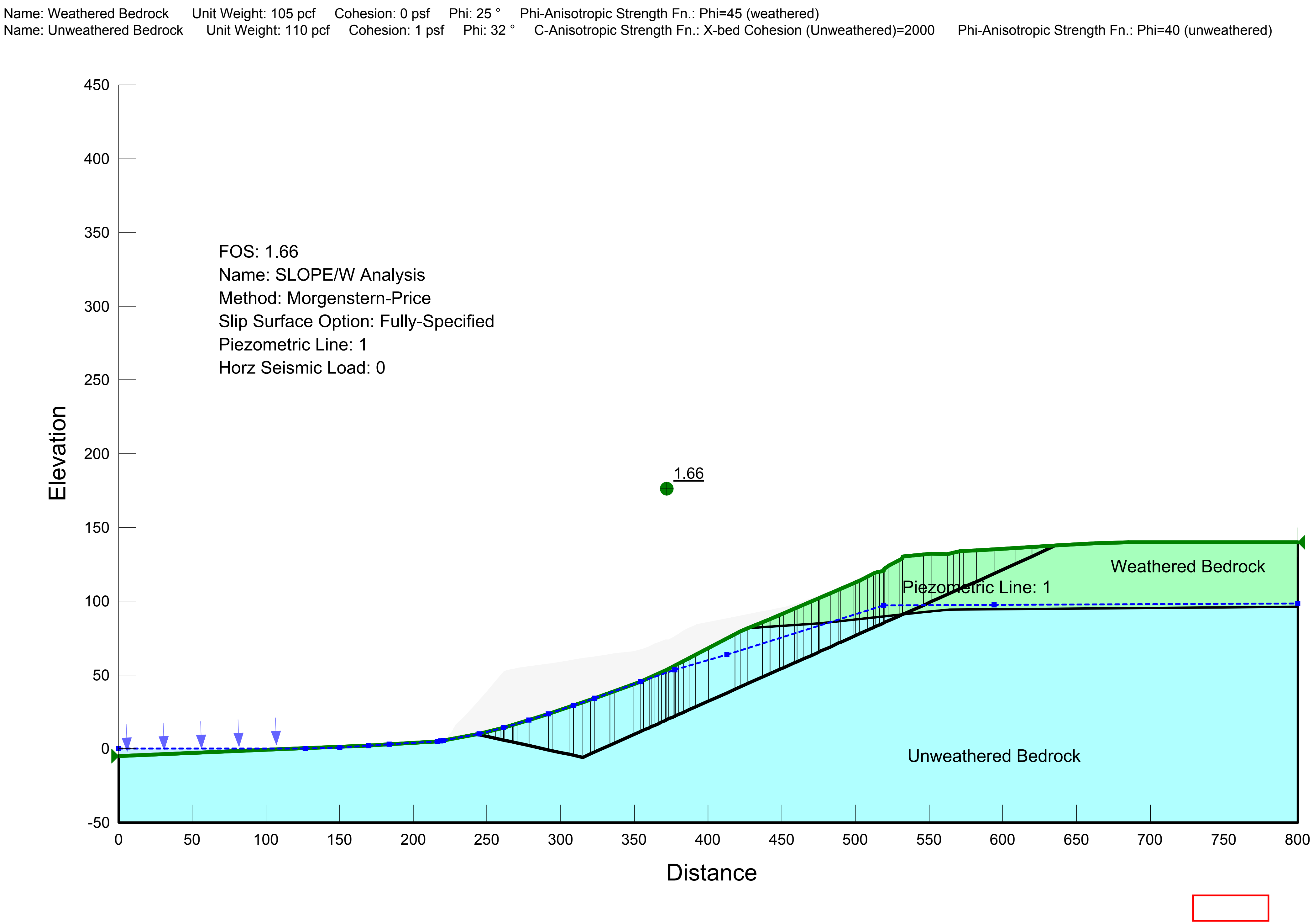
Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
FOS: 1.01
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
1.01
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
Distance
450
400
350
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800

Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °

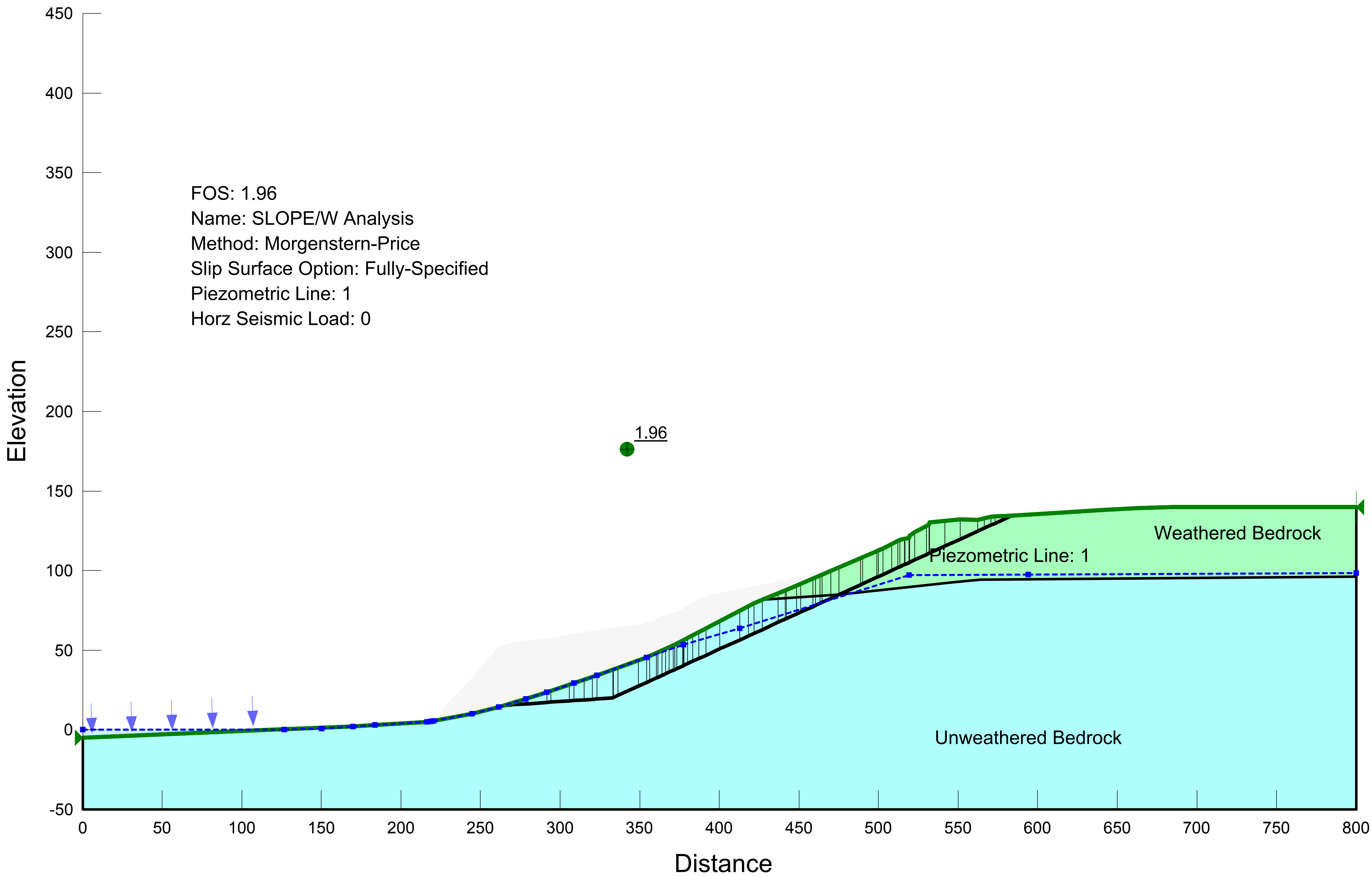




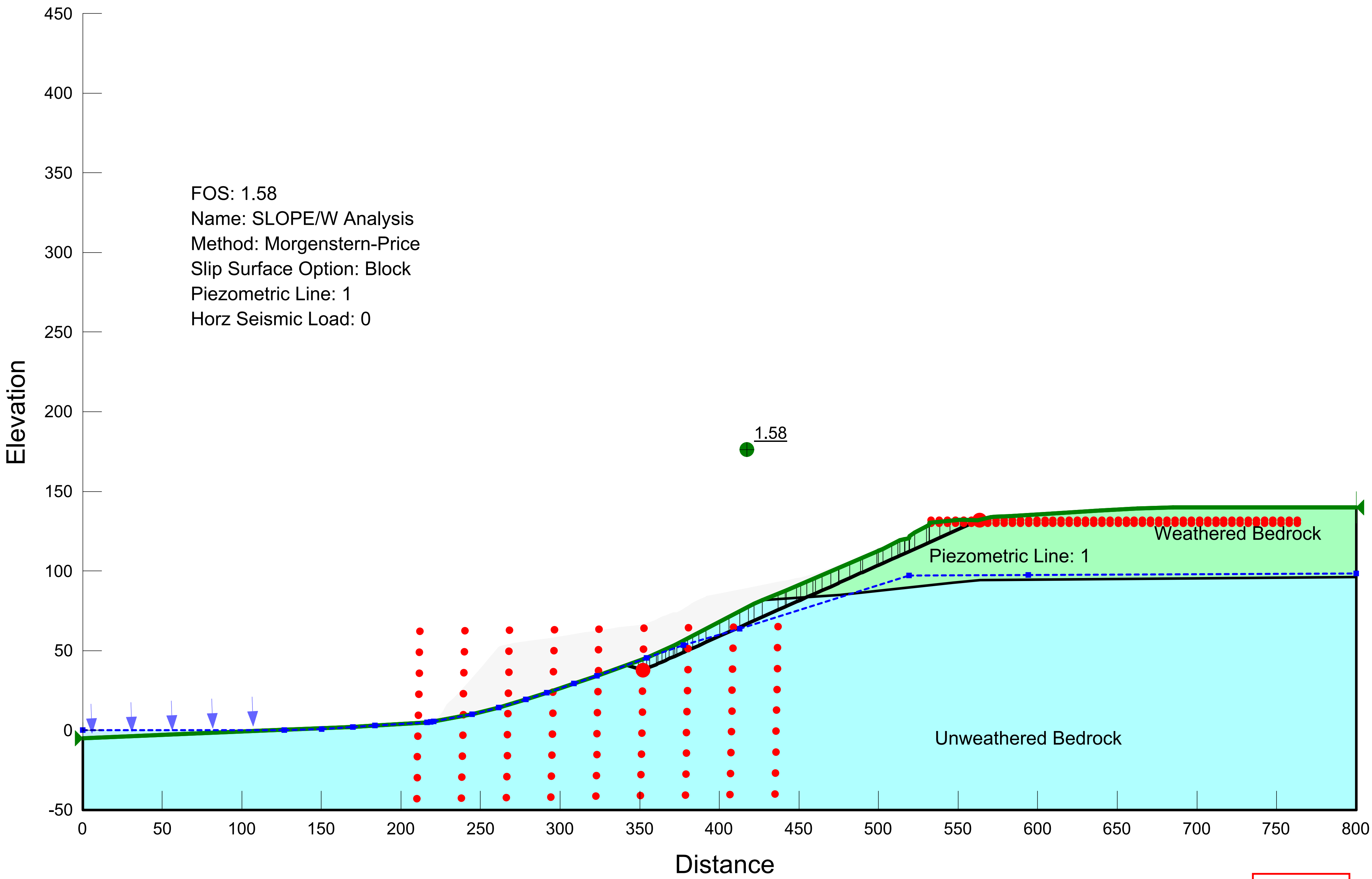
Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
FOS: 2.41
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
2.41
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
Distance
450
400
350
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800

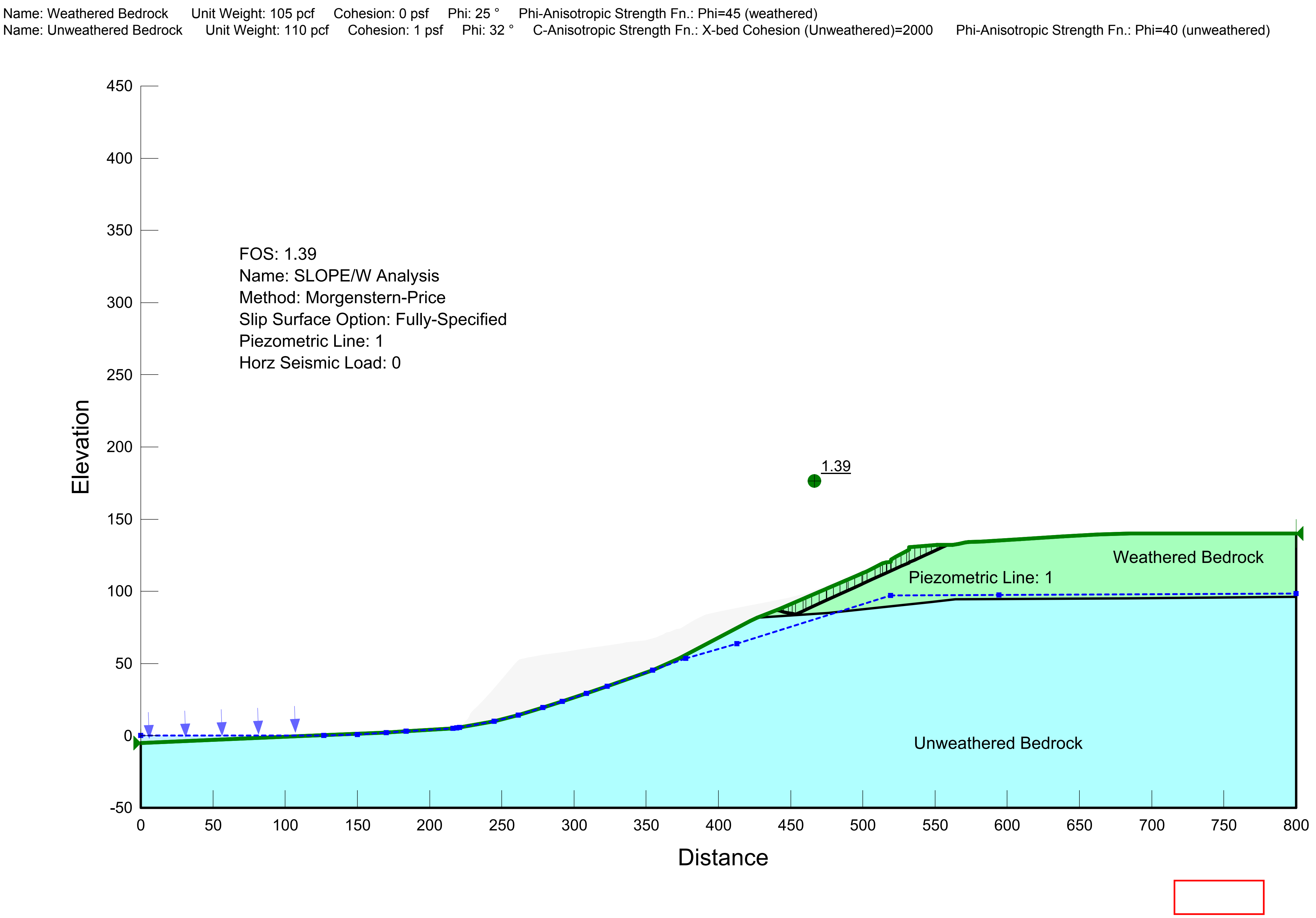

Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
FOS: 1.66
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
1.66
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
450
400
350
300
250
200
150
100
50
0
-50
Distance
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800

Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)



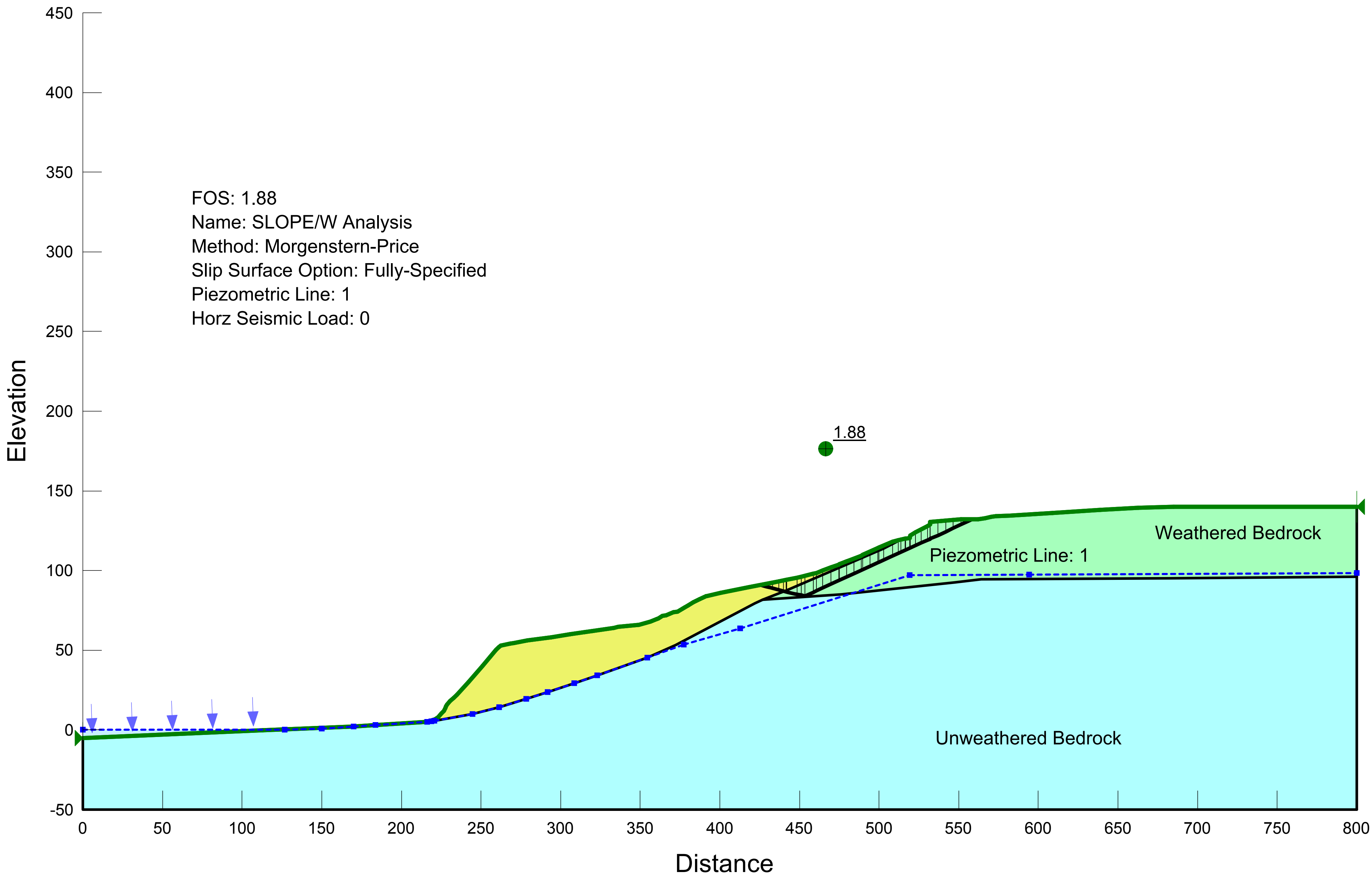
Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)

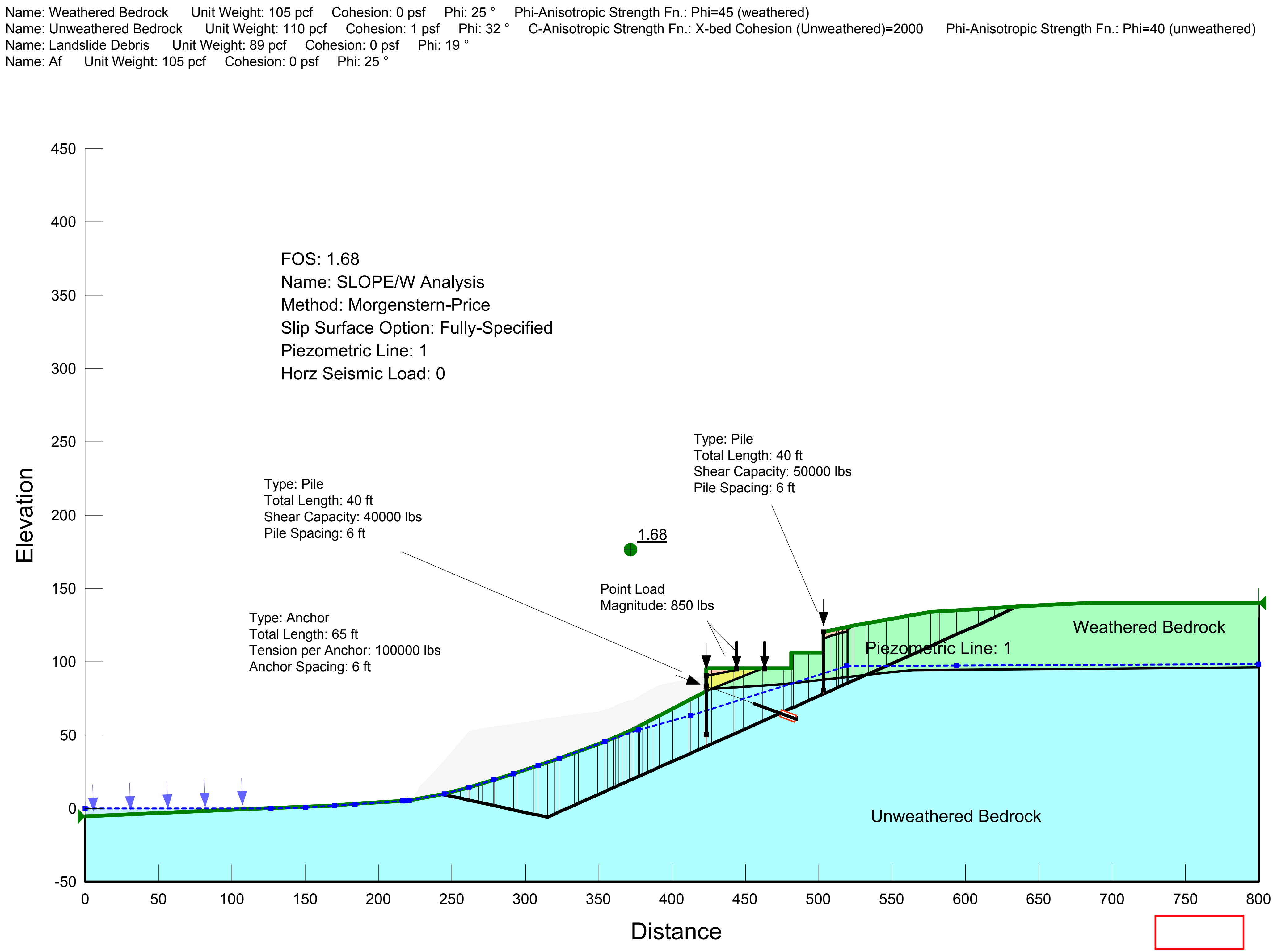




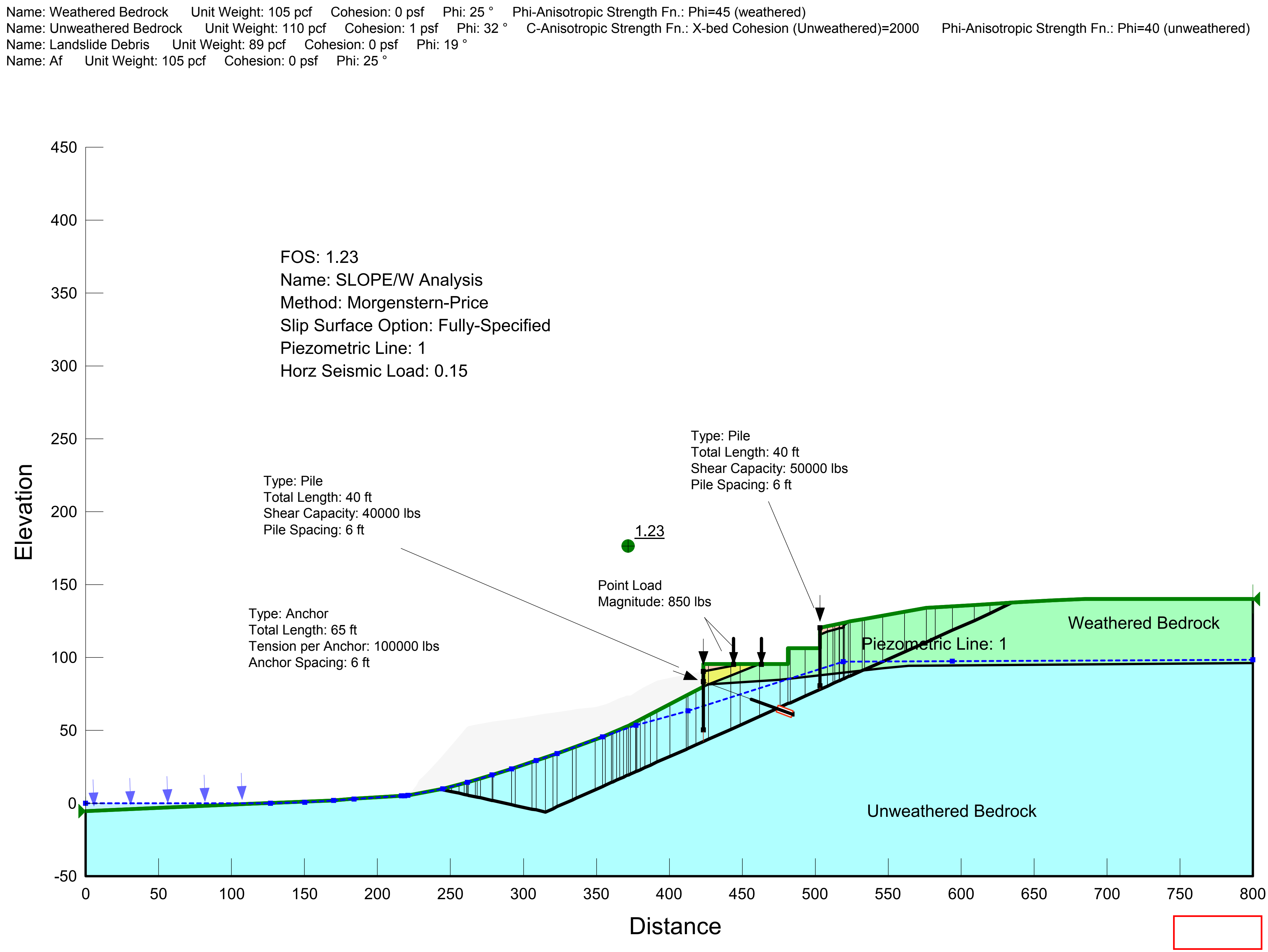
Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
FOS: 1.39
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
1.39
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
Distance
450
400
350
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800

Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °





Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
Name: Af Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 °
FOS: 1.68
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
Type: Pile
Total Length: 40 ft
Shear Capacity: 50000 lbs
Pile Spacing: 6 ft
Type: Pile
Total Length: 40 ft
Shear Capacity: 40000 lbs
Pile Spacing: 6 ft
1.68
Point Load
Magnitude: 850 lbs
Type: Anchor
Total Length: 65 ft
Tension per Anchor: 100000 lbs
Anchor Spacing: 6 ft
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
450
400
350
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
Distance


Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
Name: Af Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 °
FOS: 1.23
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0.15
Type: Pile
Total Length: 40 ft
Shear Capacity: 50000 lbs
Pile Spacing: 6 ft
Type: Pile
Total Length: 40 ft
Shear Capacity: 40000 lbs
Pile Spacing: 6 ft
1.23
Point Load
Magnitude: 850 lbs
Type: Anchor
Total Length: 65 ft
Tension per Anchor: 100000 lbs
Anchor Spacing: 6 ft
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
450
400
350
300
250
200
150
100
50
0
-50
0
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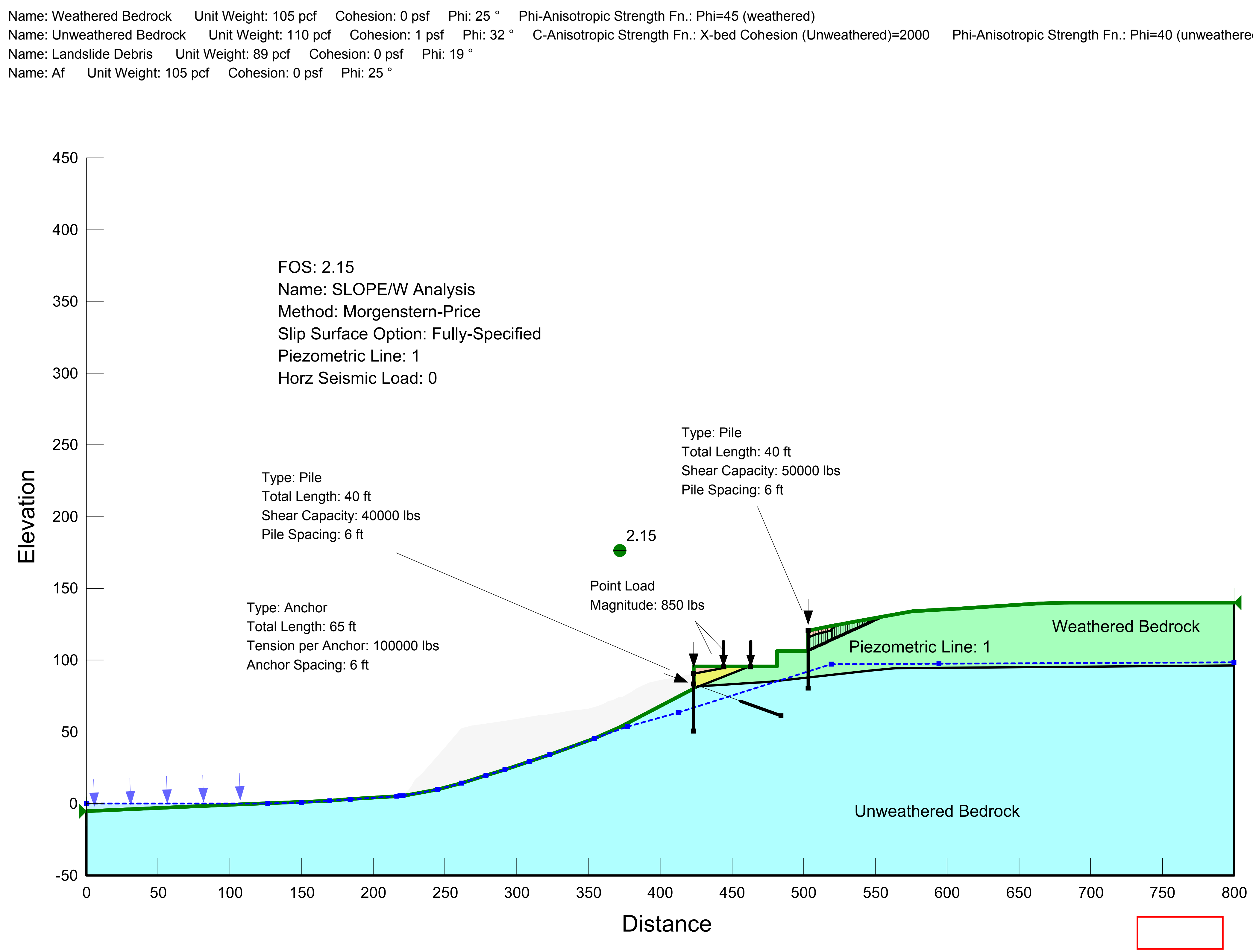
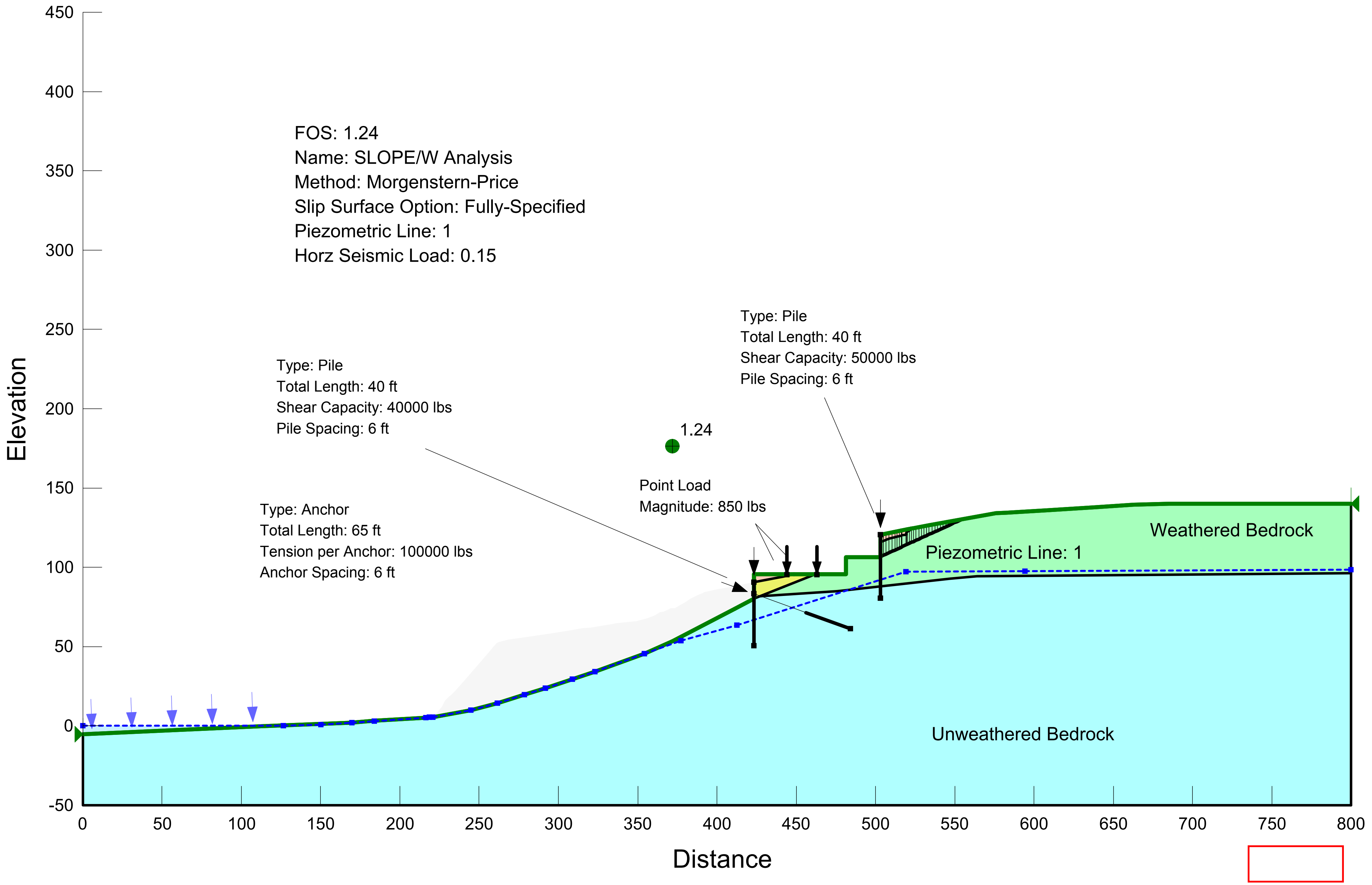
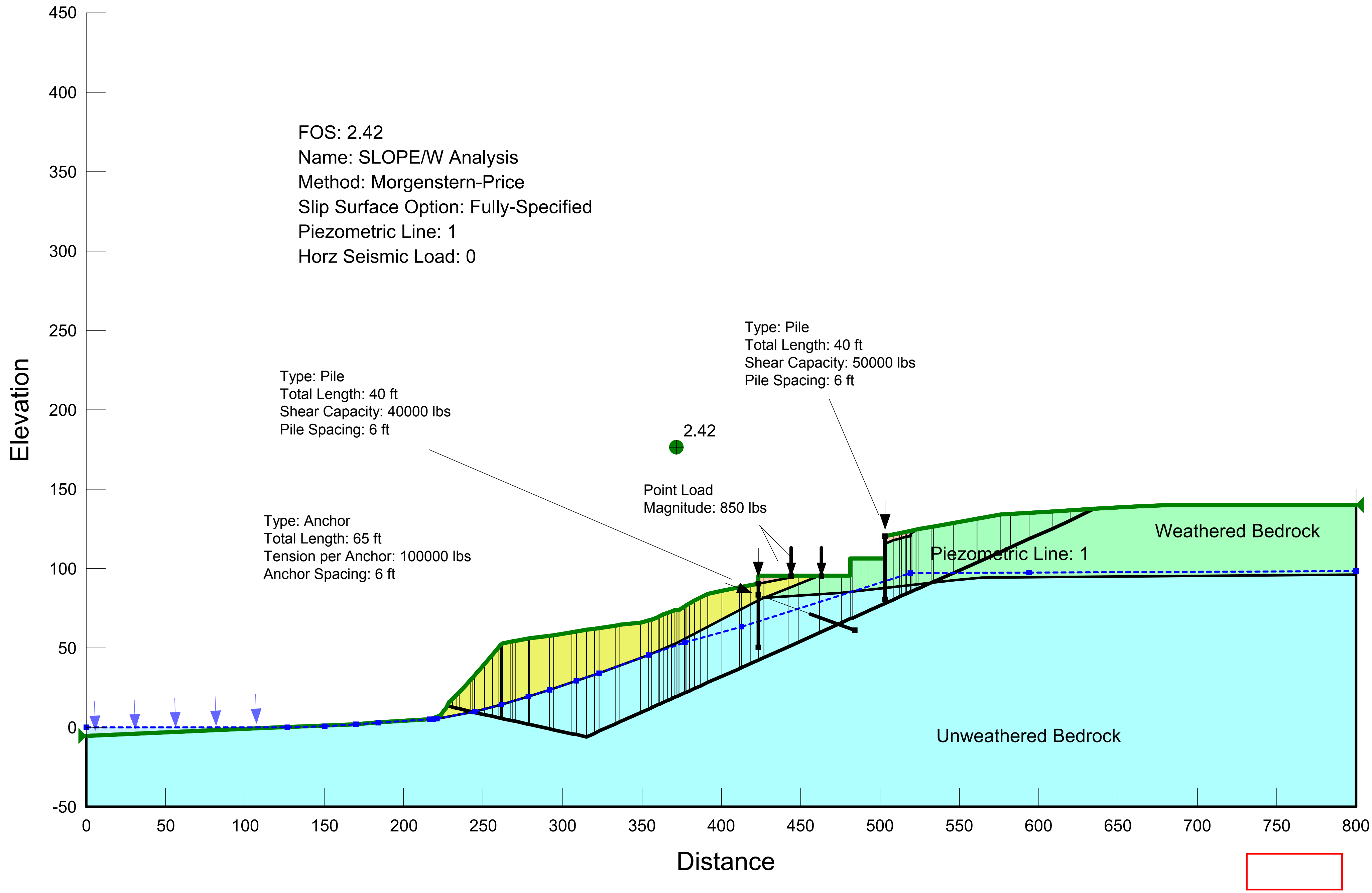
700
750
800
Distance


Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathere
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
Name: Af Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 °
FOS: 2.15
Name: SLOPE/W Analysis
Method: Morgenstern-Price
Slip Surface Option: Fully-Specified
Piezometric Line: 1
Horz Seismic Load: 0
Type: Pile
Total Length: 40 ft
Shear Capacity: 50000 lbs
Pile Spacing: 6 ft
Type: Pile
Total Length: 40 ft
Shear Capacity: 40000 lbs
Pile Spacing: 6 ft
2.15
Point Load
Magnitude: 850 lbs
Type: Anchor
Total Length: 65 ft
Tension per Anchor: 100000 lbs
Anchor Spacing: 6 ft
Weathered Bedrock
Piezometric Line: 1
Unweathered Bedrock
Elevation
450
400
350
300
250
200
150
100
50
0
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
Distance

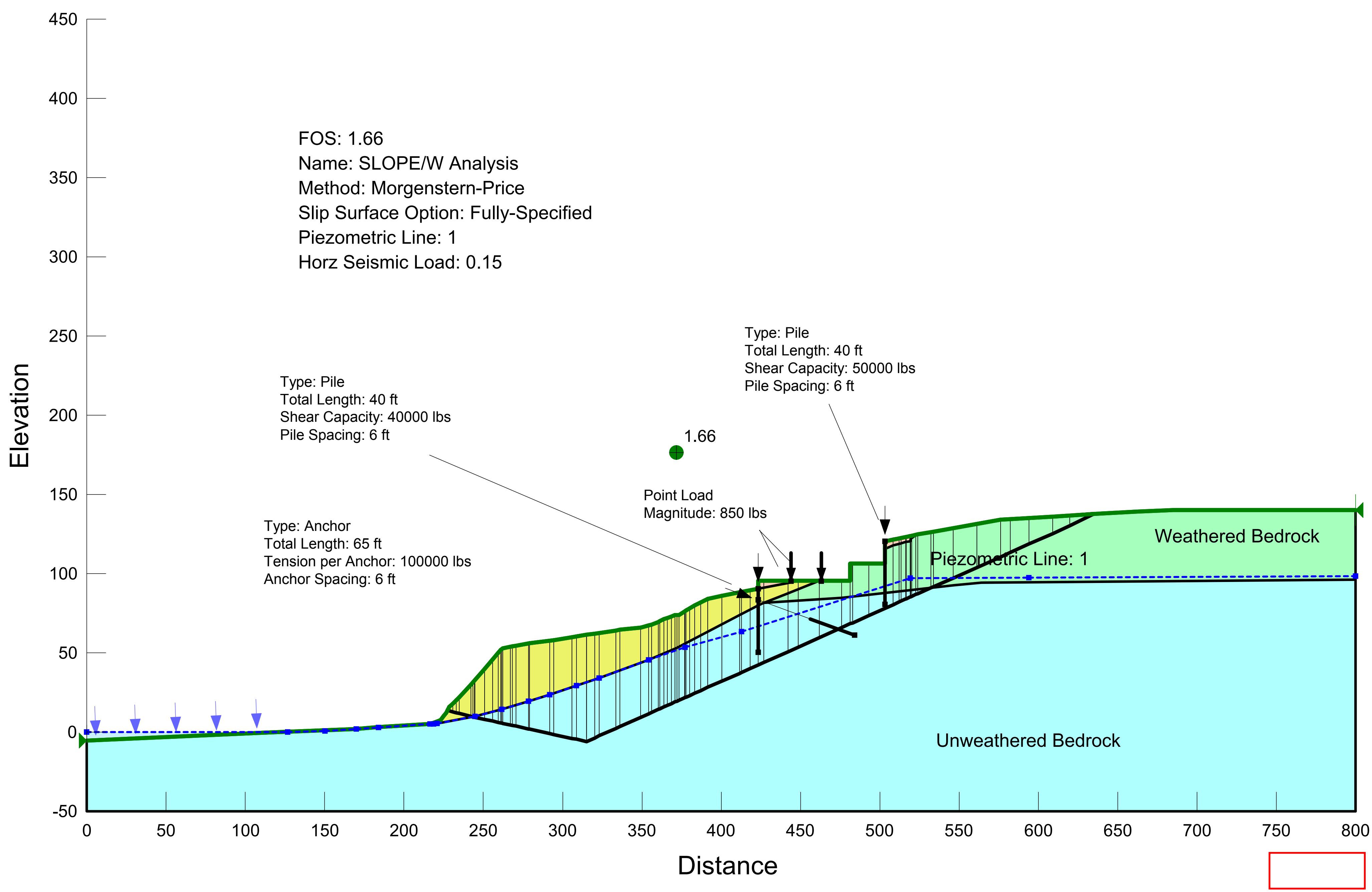
Name: Weathered Bedrock   Unit Weight: 105 pcf   Cohesion: 0 psf   Phi: 25 °   Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock   Unit Weight: 110 pcf   Cohesion: 1 psf   Phi: 32 °   C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000   Phi-Anisotropic Strength Fn.: Phi=40 (unweathere
Name: Landslide Debris   Unit Weight: 89 pcf   Cohesion: 0 psf   Phi: 19 °
Name: Af   Unit Weight: 105 pcf   Cohesion: 0 psf   Phi: 25 °



Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
Name: Af Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 °



Name: Weathered Bedrock Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 ° Phi-Anisotropic Strength Fn.: Phi=45 (weathered)
Name: Unweathered Bedrock Unit Weight: 110 pcf Cohesion: 1 psf Phi: 32 ° C-Anisotropic Strength Fn.: X-bed Cohesion (Unweathered)=2000 Phi-Anisotropic Strength Fn.: Phi=40 (unweathered)
Name: Landslide Debris Unit Weight: 89 pcf Cohesion: 0 psf Phi: 19 °
Name: Af Unit Weight: 105 pcf Cohesion: 0 psf Phi: 25 °



EXPLANATION

Earth Materials

Qt — Quaternary Terrace Deposits

Tm — Monterey Formation

Als — Active Landslide

Map Symbols

SI-5 — Small-diameter boring with inclinometer casing and piezometers installed by Cotton, Shires and Associates, Inc. in May 2011

LD-3 — Large-diameter boring drilled and logged by Cotton, Shires and Associates, Inc. in October 2011

DH-6 — Large-diameter boring drilled by Padre Associates, Inc. in September 2005 (DH-1 and DH-2) and June 2006 (DH-3 through DH-6)

B-3 — Large-diameter boring drilled by Campbell Geo, Inc. in December 2006

Stratigraphic bedding orientations collected by Cotton, Shires and Associates, Inc.

Average stratigraphic bedding orientations collected during the logging of large-diameter borings

Bedding orientation on landslide basal rupture surface collected in large-diameter boring

70' — CSA 10' Contour

CSA 2' Contour

CSA 10' Contour (Approximate)

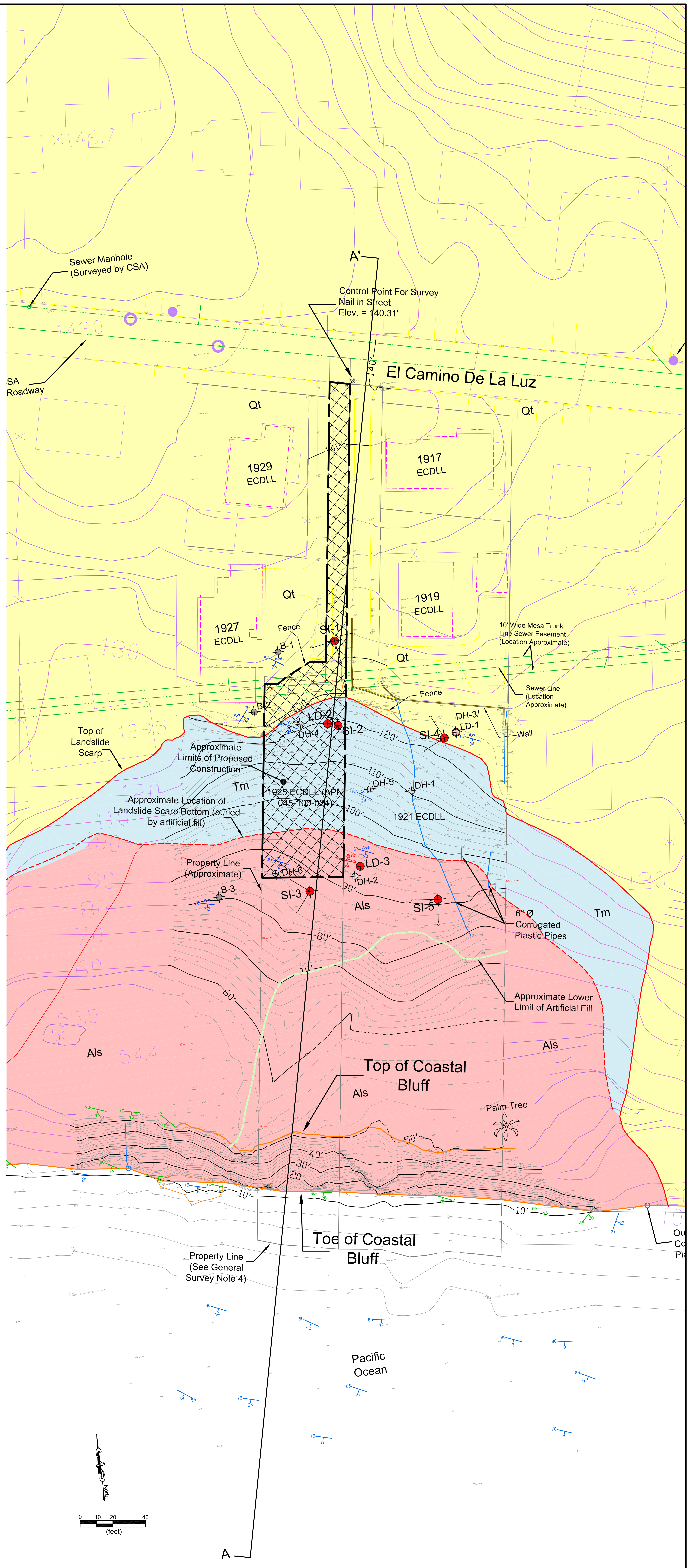
CSA 2' Contour (Approximate)

CSA Survey Point

60 — City of Santa Barbara 10' Contour

City of Santa Barbara 2' Contour

All Lines of This Color Indicate Features From City of Santa Barbara Map, Including, but not Limited to: Houses, Fences, Roads, Vegetation and Power Poles



SURVEY LIMITATIONS NOTES

1. This is not a map of a boundary survey. No property corners have been set as part of this work.

2. Survey monuments found in the course of this mapping are set by others, and have been used only as a reference for the purpose of topographic mapping, without our verification of their agreement with applicable legal descriptions and seniority of deeds.

3. Relation of topographic features (i.e., fences, walls, trees, power poles, etc.) to property lines as shown on this map is subject to the adjustments that a boundary survey may require.

4. This survey was prepared without the benefit of a Title Report. Easements, if shown, should be considered approximate in location.

5. If this map is provided in an electronic format as a courtesy to client, delivery of the electronic CAD file does not constitute delivery of a professional work product. The signed paper print delivered with this electronic CAD file constitutes our professional work product and, in the event the electronic CAD file is altered, the print must be referred to for the original and correct survey information. We shall not be responsible for any modifications made to the electronic CAD file or for any products derived from the electronic CAD file which are not reviewed, signed and sealed by us.

General Survey Notes

1) All dashed lines on this map represent features (houses, walls, topography, etc.) that have not been surveyed by Cotton, Shires and Associates and are approximate only.

2. Vertical Datum for CSA topography based on NOAA published value for mean lower low water (MLLW) in Santa Barbara.

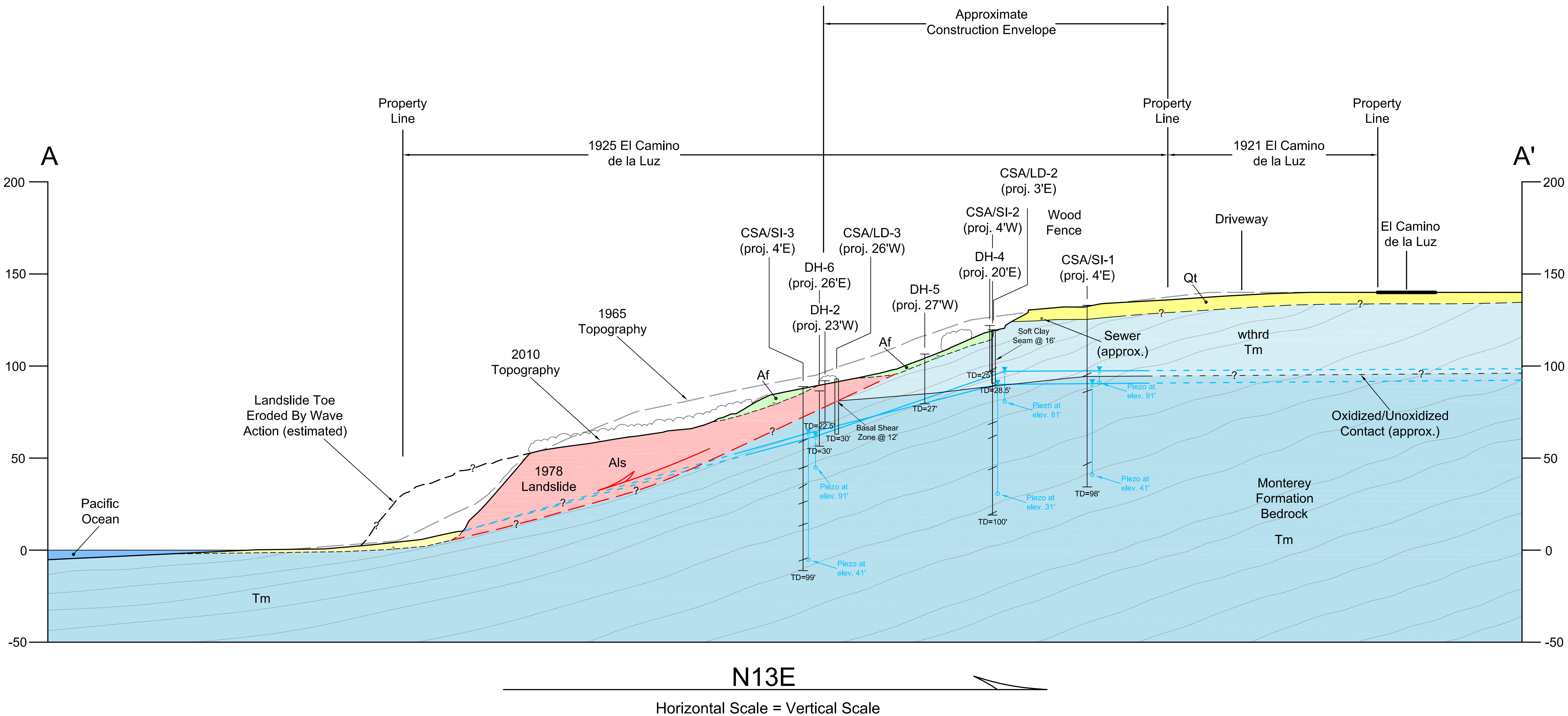
3. City of Santa Barbara topography and features taken from map dated 4/10/95 (Revised April 1997) from County of Santa Barbara website (http://www.countyofsb.org/pwd/water/TopoFloodControl1.htm).

4. Southern property lines are based on the MHTL elevation of 4.63 feet above MLLW (MHTL from David Skelly, GeoSoils, Inc., "Wave Runup & Coastal Hazard Analysis, 1921 El Camino de la Luz & 1925 Camino de la Luz, Santa Barbara, Santa Barbara County, California").

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Site Plan and Geologic Map
1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
AM/TPS	1"= 30'	G0058
APPROVED BY	DATE	PLATE NO.
JW	OCTOBER 2012	1



COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Engineering Geologic Cross Section A-A'
1925 El Camino De La Luz
APN:045-100-024
SANTA BARBARA, CALIFORNIA

GEO/ENG BY	SCALE	PROJECT NO.
JD	1"= 60'	G0058
APPROVED BY	DATE	PLATE NO.
JW	OCTOBER 2012	2